EVALUATION OF ONSITE WASTEWATER TREATMENT AND DISPOSAL:
DEMONSTRATION OF ALTERNATIVE TECHNOLOGIES
FINAL REPORT VOLUME II

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FINAL REPORT

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Executive Summary

According to the 1990 U.S. Census, about 17,800 conventional septic tank drainfield systems have been installed in Bernalillo County, New Mexico (Gaume et al., 1995). Although the extent of groundwater contamination caused by these systems is not clear, recent studies indicate that groundwater contamination caused by the septic tank drainfield systems may be widespread (McQuillan et al., 1989; Kues, 1990; Kues et al., 1995). Although conventional septic tank drainfield systems are widely used in the U.S. for onsite wastewater disposal, groundwater contamination resulting from these systems has been observed in many areas (Gover, 1996; Earp et al., 1986). While the conventional septic tank drainfield systems may remove considerable amounts of BOD, TSS, and fecal coliform, they have little ability to remove nitrogen species completely. In general, the effluents of septic tanks contain large amounts of NH_4^+ and very little of NO_3^-. Oxidation of NH_4^+ to NO_3^-, however, occurs quickly in the vadose zone underneath the drainfield, which causes a high concentration of NO_3^- in the groundwater (Wilhelm et al., 1996). There is growing concern regarding the impairment of groundwater supplies by unregulated onsite wastewater treatment systems. Based upon the Albuquerque/Bernalillo County Ground-Water Protection Policy and Action Plan (GPPAP), a two-year field study was conducted to demonstrate the ability of alternative onsite systems to remove conventional wastewater pollutants, particularly nitrogen species (organic-N, NH_4^+, and NO_3^-) (Thomson et al., 1996).

This study compiled the information developed in the previous studies and implemented a field test program to support the development of new onsite wastewater ordinances for the Bernalillo County Environmental Health Department and the City of Albuquerque. The study was intended to obtain both qualitative and quantitative data to support the ordinance development. This involved both technical and scientific approaches as well as gathering information based on observations, surveys, and the experience of the investigative team. The basic objectives for the Phase II study were:

- Propose guidelines for installing, maintaining and reviewing and inspecting liquid waste systems;
- Establish expected treatment efficiencies of each unit type;

- Recommend a procedure for evaluating systems to determine if they meet the performance standards;
- Determine how the efficiency of the units is affected by the method of installation, operation and maintenance procedures, and/or inspection procedures;
- Make comparison between treatment efficiency documented in manufactures' literature and expected treatment efficiency in the field; and
- Make recommendations concerning the skills and training needed to properly install and maintain the systems.

In order to accomplish the goals and objectives of this research a multitask approach was employed. This involved field-testing selected technologies, reviewing pertinent literature and manufacturer data, surveying installers and homeowners, interviews with regulatory personnel, attendance of conferences and workshops, and in-depth reviews of other state's existing or proposed regulations. This data was synthesized to form the basis for the recommendations developed in this report.

Five onsite wastewater treatment systems were evaluated for field study. The systems selected were three commercial systems: Whitewater, Clearstream, FAST, a conventional septic tank, and a SSF constructed wetlands. The systems were installed by local licensed installers at various locations in Bernalillo County, New Mexico. The systems were installed at typical three bedroom residences and were owner operated with no assistance from the research team. All sites were modified to allow sampling of system influent and effluent wastewaters and flowrates. The systems were evaluated by conducting a comprehensive sampling and analysis program. This program used a "hands off" approach that depended on maintenance provided by the installer, the manufacturer's representative, or the homeowner to operate and adjust the system. The first part of the plan was to systematically sample the influent and effluent for each system. This was performed about every two weeks or twice per month over the course of the study period. The quantity of samples taken provided a sufficient statistical basis to evaluate the performance of each system. The use of statistics provided a sound scientific justification for the interpretation of the observed results. This ongoing sample regime allowed the research team to observe possible startup problems, seasonal variations or variations in operation due to water softeners or system component failure.

Samples were collected using a modified compositing approach. A sample sump with a fixed volume was put online the day sampling began. This sump collected wastewater as it flowed through the sump over a prescribed period of time. Typically the sump was set up to collect a sample from about four PM until about eight AM the following morning. The wastewater flow was allowed to fill the sump and overflow to the unit or discharge to the disposal field. The incoming wastewater mixed with wastewater from the previous event and as each event occurred, this mixing continued. Some washout, loss, and dilution effects from previous events were expected, but a consistent and flow equalized sample was collected at the end of the sample collection period. These samples were homogenized by pumping the sump contents within the sump using a Vortex type sewage pump to macerate solids and provide uniform particle sizes. Samples were taken, preserved, and transported to the Albuquerque City Laboratory for analysis. Field analyis was conducted onsite in the sump prior to pumping. General observations regarding odors etc. were also noted.

Each water quality parameter result for influent and effluent was analyzed for a suite of basic statistical parameters that included the mean, standard deviation, percent error, maximum and minimum values, etc. to provide basic information about the data variability. The influent and effluent parameters were compared using a one way analysis of variance (ANOVA) to compare treatment means. This allowed the team to determine comparative properties and performance of water quality category for influent and effluent. This analysis was performed for all systems. Some systems did not operate continuously throughout the study period and statistics were not valid on these systems, this was noted in the discussion. Finally, the results from each system were compared to the proposed onsite treatment performance standards. This allowed relative comparisons to be made without directly determining statistics for the comparisons. Direct statistical analysis between systems with such differing influent and effluent values would have been difficult to statistically justify.

The Whitewater system, a sedimentation unit and a suspended-growth, aerobic biological process, was tested for 65 weeks. This was an off the shelf unit without a pretreatment trash tank. The system was hydraulically tested, exhibited no short-circuiting and had near complete-mix flow, flow pattern. It was difficult to determine from unit operations perspective, how this system achieved denitrification. The system influent and effluent data is shown in Tables A and B for concentrations of BOD_5, COD, TSS, FC, ammonia, and TN, respectively. This system achieved an average percent removal of 71.6, 59, 98, 91, 49.4, and 68.3 % for BOD_5, COD, TSS, FC, ammonia, and TN, respectively. The best effluent values for this system were 36.3 mg/L, 181.9 mg/L, 8.4 mg/L, 2.7×10^4 cfu/100mL, 1.5 mg/L, and 14.2 mg/L for BOD_5, COD, TSS, FC, ortho-P, and TN, respectively. Operationally the system had problems with the supply of air to the process and the aeration fittings appeared to be weakness in the system.

Table A Summary of Mean Influent Data for the Five Test Systems.

Parameter/System	**Whitewater**	**Clearstream[1]**	**Septic Tank**	**FAST**	**Wetlands**
BOD_5, mg/L	330	N/A	1,147	523	208
COD, mg/L	914	N/A	2,404	1,842	544
TSS, mg/L	1,296	N/A	2,232	1,356	41
NH_3-N, mg/L	28	N/A	54	49	70
TN, mg/L	62	N/A	124	69	77
FC, cfu/100mL	6.02×10^5	N/A	4.6×10^5	6.02×10^5	3.43×10^5
Ortho-P, mg/L	4.9	N/A	6.4	10.1	6.7

[1]Very few data values, see section on Clearstream for summary.

Table B Summary of Mean Effluent Data for the Five Test Systems.

Parameter/System	**Whitewater**	**Clearstream[1]**	**Septic Tank**	**FAST**	**Wetlands**
BOD_5 , mg/L	93.5	N/A	150.6	76.0	96.0
COD , mg/L	374.5	N/A	421.8	882	310.0
TSS , mg/L	26.2	N/A	49.2	28.3	16.7
NH_3-N , mg/L	13.9	N/A	44.4	16.0	39.6
TN, mg/L	19.7	N/A	59.5	29.0	43.2
FC, cfu/100mL	5.25×10^4	N/A	7.3×10^4	2.04×10^4	1.48×10^4
Ortho-P, mg/L	3.0	N/A	6.4	3.3	4.8

[1]Very few data values, see section on Clearstream for summary.

The submerged surface flow constructed wetlands system, a fixed-film, anoxic/aerobic/anaerobic with emergent plants biological process, was tested for 65 weeks. The SSF cell of the system was hydraulically tested and exhibited no short circuiting and a dispersed plug-flow, flow pattern. The system influent and effluent data is shown in Tables A and B for concentrations of BOD_5, COD, TSS, FC, ammonia, and TN, respectively. This system achieved an average percent removal of 53.7, 42, 45, 95.6, 43.5, and 43.8 % for BOD_5, COD, TSS, FC, ammonia, and TN, respectively. The best effluent values for this system were 23.6 mg/L, 97.8 mg/L, 2 mg/L, 1.31×10^3 cfu/100mL, 1.5 mg/L, and 14.2 mg/L for BOD_5, COD, TSS, FC, ortho-P, and TN, respectively. Operationally the system worked very well and for much of the summer months the wetland cell produced no discharge. The overall system with a septic tank, two-stage wetlands (SSF and SF cells) operated at a zero discharge mode throughout the study period.

The Clearstream system, a suspended-growth, aerobic biological process with recirculation to an anoxic suspended growth chamber, was operated only intermittently as the homeowner moved out in the early part of the study. This unit was a custom designed system. The system was not hydraulically tested and insufficient data was collected to evaluate the system.

The Cofast or FAST system, a suspended-growth combined with fixed film, aerobic/anoxic, biological process, was operated for 55 weeks. This was an off the shelf unit without a pretreatment trash tank. The system was hydraulically tested and exhibited no short circuiting and had a near complete-mix flow, flow pattern. The system influent and effluent data is shown in Tables 1 and 2 for concentrations of BOD_5, COD, TSS, FC, ammonia, and TN, respectively. This system achieved an average percent removal of 85.5, 52.1, 97.9, 99, 67.5, and 75.5 % for BOD_5, COD, TSS, FC, ammonia, and TN, respectively. The best effluent values for this system were 60 mg/L, 593 mg/L, 8.8 mg/L, 2.0×10^2 cfu/100mL, 1.6 mg/L, and 13.3 mg/L for BOD_5, COD, TSS, FC, ortho-P, and TN, respectively. Operationally the system had problems with the supply of air to the process, the PVC pipe supplying air was not properly glued and leaked decreasing the air applied during a portion of the study.

The septic system, a sedimentation unit and anaerobic biological process, was operated for 35 weeks over the study period. This was an off the shelf unit without a pretreatment trash tank. Two attempts were made to hydraulically test the unit, but failed due to insufficient flow. The occupant appeared to be out of town frequently or worked away from the home for extended periods. The system influent and effluent data is shown in Tables A and B for concentrations of BOD_5, COD, TSS, FC, ammonia, and TN, respectively. This system achieved an average percent removal of 86.9, 82, 97.8, 72, 17, and 52 % for BOD_5, COD, TSS, FC, ammonia, and TN, respectively. The best effluent values for this system were 148.6 mg/L, 246 mg/L, 32.6 mg/L, 3.28×10^4 cfu/100mL, 1.5 mg/L, and 45 mg/L for

BOD_5, COD, TSS, FC, ortho-P, and TN, respectively. Operationally the system was typical for septic tanks systems, but exhibited a very low overall flow.

A summary of the operational parameters and loadings for all the systems is shown in Table C. This data indicates that all systems tested were hydraulically underloaded with respect to design assumptions. Note that for all processes the actual HRT was much longer than the design values. These longer values can impact the performance of all systems tested. While loadings for BOD, TSS, and TN vary, they are similar despite the wide range of households tested under this study.

Table C Summary of Operational Data for the Five Test Systems.

Parameter/System	Whitewater	Clearstream	Septic Tank	FAST	Wetlands[1]
Design Flow, gpd	500	N/A	450	450	450
Design HRT, days	1.81	N/A	2.7	1.8	2.7
Actual Flow, gpd	161.8	N/A	52.7	119.6	201.1
Actual HRT, days	5.6	N/A	23.0	6.8	8.0
BOD Loading, lbs/day	0.52	N/A	0.50	0.52	0.34
TSS Loading lbs/day	1.74	N/A	0.98	1.35	0.05
TN Loading lbs/day	0.08	N/A	0.05	0.12	0.13

[1] Loadings do not include raw waste but effluent from a pretreatment septic tank.

The systems tested in this study did not appear to be capable of meeting a 10 mg/L TN standard for effluent discharges to the subsurface. This does not mean there are no systems that can do this, but under the conditions used in this study none of the test sites were able to meet this standard. In order to the meet the standard for TN, systems should be optimized for N removal and carbon removal rather than just carbon removal as is the case for most systems currently used. The business of onsite wastewater treatment system is very dynamic and several states are adapting nitrogen limits in their regulations and ordinances. Given time to develop, new, lower-cost, technologies will be commercially available that will meet more stringent standards.

Aeration systems (blowers or compressor, airlines, and diffusers) associated with the mechanical aerobic units were a noticeable weakness and the partial or full failure of this part of the system resulted in very poor system performance. Rigorous inspections must be performed and air supply units pressure-tested to assure their operability.

Systems must be evaluated based on both the theory of operation and manufacturer supplied test data. In many cases the theory of operation is largely ignored and the claims published by the manufacturer are the only substantial information available. Test data must be collected in a systematic fashion to allow for the analysis of the data. With nitrogen, the complete species must be given not just nitrate or ammonia or TKN. The conditions (composite grab or otherwise) under which the data was taken and the flow must be specified. Without information of this kind, rational judgement cannot be made about the ability of a system to meet the required standards.

Surveys conducted on sample populations of installers and homeowners in the Bernalillo County area provided interesting results on a number of topics. Significant results from the installers survey indicated that Septic tanks accounted for nearly 98% of all installations and alternative systems accounted for only 2% of the installations. Installers indicated the most common reason for an installation was new construction (75%), followed by system failure (19%), and remodeling (6%). Leachfield failures were the most common reason for repairs and poor installation or drainfield undersizing cited as the most common reason for failure. Eighty nine percent of the installers thought the quality of septic tanks they used was good, but 75% thought a septic tank certification program would benefit their business. Seventy six percent of the installers were aware of the efforts by the city and county to improve the onsite wastewater ordinances. Homeowners surveyed indicated that 68% of the respondents had maintenance performed on their systems and 53% had their septic tanks pumped every 6 to 12 months. But 39% had never had their systems serviced or pumped. Finally, 74% percent of those homeowners surveyed thought the county was doing a good job at protecting drinking water supplies and the environment.

Finally, based on this study and the data gathered from other states, various literature sources, and previous phases of work, guidelines covering the use of alternative systems were developed. These guidelines provided specific recommendations pertaining to lot sizes, site evaluations, system performance standards (technology-based standards and best management practices (BMP), system maintenance guidelines, system installation, and inspection of new and existing systems.

Chapter 6 - Introduction

More than half the nation's population (53 percent) and nearly all of its rural population (97 percent) rely on groundwater for drinking water. Of this groundwater, 40 percent goes to public water supply withdrawals and 40 percent for domestic and commercial use. According to Moody (1990), in 1980 about 22 million domestic disposal systems were in operation and about one-half million new systems are installed each year. It is estimated that from one-third to one-half of existing systems could be operating improperly because of poor location, design, construction, or maintenance practices. Contamination caused by septic system failure will probably increase because the 10 to 15 year design-life of systems built in the 1960's and 1970's is now exceeded. Contaminants from septic systems may include bacteria, nitrate, dissolved solids, chlorides, iron, and organic substances including synthetic organic chemicals.

The standard onsite wastewater treatment system in the US consists of a septic tank followed by a soil adsorption system. Septic tanks contribute the largest volume of wastewater, 800 billion gallons per year to the subsurface, and are the most frequently reported cause of groundwater contamination associated with disease outbreaks (Yates 1985). The consumption of untreated or inadequately treated groundwater was responsible for over one-half of all the waterborne outbreaks and 45 percent of all cases of waterborne disease in the US from 1971 to 1979. Overflow or seepage of sewage from septic tanks or cesspools was responsible for 43 percent of the outbreaks and 63 percent of the cases of illness caused by the use of contaminated groundwater. Groundwater contamination resulting directly from septic tanks systems has been observed in many areas of the US including Nassau County, Long Island (Yates 1985), and Albuquerque, New Mexico (Earp and Koschal 1988). Septic tanks in New Mexico discharge 51 million gallons of wastewater into the groundwater each day.

The septic tank is usually a two-chambered tank designed to separate, hold, and degrade collected solids, separate, and retain floatable scums, oils, and greases from the wastewater stream. The soil adsorption system may consist of a trench, bed, or pit; sized and placed in the soils and backfilled with drain field rock and original soil materials. These systems, when properly installed taking into account climatic, soil, and topography considerations, can be expected to remove considerable amounts of five day biochemical oxygen demand (BOD_5), total suspended solids (TSS), and fecal coliform bacteria found in typical domestic wastewaters (Tyler et al., 1977). However, it is estimated that these systems effectively remove 10 to 50 percent of the total nitrogen species including organic nitrogen, ammonia, and nitrate (USEPA 1980; Laak 1986). Ammonia and organic nitrogen appear to be readily converted via nitrification in the soil absorption system to nitrate. Nitrate discharged to groundwater from traditional onsite septic tank/absorption field systems, are presumed to be controlled by simple dilution and no further transformations or removal can be expected. Additional problems as noted by Woessner and Ver Hey (1987) suggest that:

- Only a small portion of the entire drainfield is used for effluent treatment resulting in inadequate detention time for biological treatment;
- Anaerobic conditions prevail in the drainfields;
- The vadose zone does not supply appreciable treatment;
- The aquifer reduces concentration of effluent by dilution.

It is becoming increasingly clear that future onsite treatment systems must be capable of removing nitrogen and that typical septic tank/leachfield installations will not meet these needs.

Nitrogen can be removed from wastewater by simple source separation of blackwater and greywater; physical/chemical methods such as ion exchange, chlorine oxidation, or reverse osmosis; and by coupled nitrification/denitrification (Whitmyer et a1., 1991). While some alternative processes have been tried such as ion exchange using clinoptilolite or reverse osmosis, most onsite treatment systems depend almost entirely on biological processes for the removal of nitrogen species. Nitrogen occurs in both organic nitrogen and urea nitrogen forms in raw wastes. Urea hydrolyzes rapidly to form ammonia and carbon dioxide in the presence of microorganisms producing the enzyme urease.

Organic nitrogen undergoes ammonification to release ammonia to the wastewater. This process can be drastically slowed by the lack of oxygen or low temperatures. The breakdown of urea to form ammonia may take just a few minutes while the release of ammonia from organic nitrogen sources may take several days or months. Thus, the nitrogen in most raw wastes is a combination of these nitrogen forms with ammonia the dominant species. Once ammonia is present it can be removed via a two-step nitrification process that consumes oxygen and alkalinity as ammonia is converted to nitrate. If either oxygen or alkalinity or both are limiting, the reaction will not proceed. In addition, the reaction is limited by temperature, salinity presence, or absence of a carbon source, and pH. The nitrification process is very sensitive to toxic substances and nitrifying bacteria have been used for bacterial-based toxicity testing procedures (Alleman 1986; Arbuklye and Alleman 1991).

The major method for nitrate removal is denitrification that is a two-step process in which the microorganisms use nitrate as an alternative electron acceptor in place of oxygen to convert nitrate to nitrogen gas. As opposed to nitrification, a relatively broad range of environmentally robust microorganisms can accomplish denitrification. Critical to the completion of this process is the availability of a carbon source. The carbon source is oxidized and donates electrons and nitrate gains electrons and is reduced to nitrogen gas. Many different compounds can serve as a carbon source, but not all compounds result in an efficient conversion of nitrate to nitrogen gas. Acetate and methanol provide some of the best conversion rates for this process. Gersberg et al., (1984) reported removal efficiencies of 97 percent for total inorganic nitrogen (ammonia and nitrate) and 94 percent for total nitrogen (TN) are achieved via denitrification with methanol. Substituting plant biomass in the form of mulch as the carbon source resulted in the removal of 95 percent total inorganic nitrogen and 89 percent TN at hydraulic loading rates of 8.4 to 12.5 cm/d. Blending primary effluent as the carbon source with the secondary effluent resulted in removal efficiencies as high as 79 percent for total inorganic carbon and 77 percent TN while maintaining 89 percent removal rates for BOD and TSS. Other types of carbon-donating materials such as peat have also been effective in the denitrification process.

Septic tanks were never designed to remove nitrogen. No operating conventional septic tank system whether properly or improperly designed, installed, or operated will meet an effluent discharge standard of TN of 10 mg/L. Movement and transformation of nitrogen under seven septic tank installations studied by Whelan and Barrow (1984), indicated that most nitrogen came from household toilets in the form of NH_4 after passing through the septic tank. After passing through the slime layer in the drainfield, the NH_4 was oxidized to NO_3, but only in the upper 0.5m of aerobic soils. Because of low cation exchange capacity of the soil and formation of NO_3, 70 to 90 percent all N from septic tanks entered the groundwater.

Nitrate is highly soluble and mobile in groundwater and is generally considered a strong indicator of potential groundwater contamination. Nitrate has been linked to cases of methemoglobinemia in infants (Bosch et al., 1 950) and, as a consequence, the United States Environmental Protection Agency (USEPA) and the World Health Organization (WHO) specify a maximum nitrate concentration in groundwater of 10 mg/L as nitrogen (Sayre 1988). Because most forms of nitrogen will eventually be converted to nitrate in subsurface disposal systems, total nitrogen has been suggested as the proper measure for onsite treatment systems. The State of New Mexico Ground Water Protection and Remediation Bureau enforces a total nitrogen standard of 10 mg/L as N or less for wastewater treatment systems discharging more than 2,000 gallons per day (gpd) to the subsurface (NM Water Quality Standards 3-103).

Although conventional septic tank drainfield systems are widely used in the U.S. for onsite wastewater disposal, groundwater contamination resulting from these systems has been observed in many areas (Gover, 1996; Earp et al., 1986). While the conventional septic drainfield systems may remove considerable amounts of BOD, TSS, and fecal coliform, they have little ability to remove nitrogen species effectively. In general, the effluents of septic tanks contain large amounts of NH_4^+ and very little of NO_3^-. Oxidation of NH_4^+ to NO_3^-, however, occurs quickly in the vadose zone underneath the drainfield, which causes a high concentration of NO_3^- in the groundwater (Wilhelm et al., 1996).

According to the 1990 U.S. Census, about 17,800 conventional septic drainfield systems have been installed in Bernalillo County, New Mexico (Gaume et al., 1995). Although the extent of groundwater contamination caused by these systems is not clear, recent studies indicate that groundwater contamination caused by the septic drainfield systems may be widespread (McQuillan et al., 1989; Kues, 1990; Kues et al., 1995). Based upon the"Albuquerque/Bernalillo County Ground-Water Protection Policy and Action Plan" (GPPAP), a two-year field study was conducted to demonstrate the ability of alternative onsite systems to remove conventional wastewater pollutants, particularly nitrogen species (organic-N, NH_4^+, and NO_3^-) (Thomson et al., 1996).

6.1 Phase II Background

The overall scope of this project was conducted in three major phases. Background on the first two phases is provided in the following sections. The primary objectives of the Phase I report were to:

- Collect information about alternative onsite wastewater treatment systems;
- Develop selection criteria for the treatment systems and potential demonstration sites; and
- Select the most promising treatment systems and potential demonstration sites.

6.2 Phase II Contract Objectives

The formal Phase II study compiled the information developed in the previous studies and implemented a field test program to support the development of new onsite wastewater ordinances for the Bernalillo County Environmental Health Department and the City of Albuquerque. The study was intended to obtain both qualitative and quantitative data to support the development of the new ordinance. This involved, providing the County with technical information, scientific information, field observation, surveys, and the experience of the investigative team. The basic objectives for the Phase II study were:

- Propose guidelines for installing, maintaining and reviewing and inspecting liquid waste systems;
- Establish expected treatment efficiencies of each unit type;
- Recommend a procedure for evaluating systems to determine if they meet the performance standards;
- Determine how the efficiency of the units is affected by the method of installation, operation and maintenance procedures, and/or inspection procedures;
- Make comparison between treatment efficiency documented in manufactures' literature and expected treatment efficiency in the field; and
- Make recommendations concerning the skills and training needed to properly install and maintain the systems.

Basic information regarding onsite systems was collected from manufacturer data, published literature, reports, and past studies by the research team (Zachritz 1994). This data was reviewed, compiled, and criterion for the performance of alternative wastewater treatment systems was developed. This criterion set effluent treatment limits for BOD, TSS, nitrate nitrogen, and ammonia nitrogen. This database included information on both experimental and commercially available treatment units. After discussion with the staff of the City of Albuquerque and Bernalillo County Department of Environmental Health the list of potential technologies was narrowed to those that met the criterion, and were commercially available. Selection criteria for a list of specific technologies was then further refined and developed. The specifics of this criterion are presented in other sections of the report. Based on the reviewed data and the selection criterion, a final list of alternative onsite treatment technologies was compiled.

Sample plans and sampling protocols from earlier studies were reviewed to assist in the development of site selection criteria and a preliminary sampling plan. General test site criteria was established. This included the type of household, number of bedrooms, and other aspects. A preliminary sample plan was proposed and a list of water quality parameters developed for the performance testing. In addition, several engineering analysis techniques were included for further analysis of the test systems. This included waste characterization studies and tracer analysis of the test systems' reactor hydraulics. Much of this information is provided in other sections of this report. The report also developed time requirements, scheduling, and cost estimates for the Phase II study.

6.3 Phase II Summary of Influent Characterization

An influent characterization study was implemented about six months prior to the beginning of the formal Phase II study, to provide background information about the type of flow patterns and wastewater strength that could be expected from some of the test households, assist in developing and refining the overall sampling test plan for the Phase II study, and provide practical information about the installation of sample sumps. Influent wastewaters for onsite treatment systems can be difficult to properly characterize because various water-use events within a dwelling create an intermittent flow of wastewater that can vary widely in strength and volume. Effluent from the system tends to be more homogeneous because all flow has undergone treatment and equalization throughout the process. A number of studies have been conducted to delineate the characteristics of wastewater produced from individual homes (Anderson, and Watson 1967; Watson et al., 1967; U.S. EPA, 1978; Laak, 1966 and 1975; Ligman et al., 1974; Siegrist et al., 1976; Metcalf and Eddy, Inc.1990). Much of this information was used to develop the USEPA Design Manual for Onsite Wastewater Treatment and Disposal Systems published in 1980.

To accomplish the objectives of this portion of the project, three single-family homesites were selected, sample access ports installed, and the sites monitored for flow and waste strength variations. Site I had two working adults and an infant, Site II had two working adults and three children, and an elderly couple occupied Site III with one working and one staying at home all day. All of the sites had garbage grinders in the sinks. Using a data acquisition system, influent from each site was sampled continuously for at least 3 days (two weekdays and one weekend) during which all of the flow was collected as individual flow volumes and their compositions were analyzed for water quality parameters.

A manhole of 3 feet in diameter and 2 to 6 ft deep depending on the site was installed at the inlet of each onsite system. Sumps with liquid volumes ranging from 7.3 to 11 gallons were installed in the manholes. A globe valve was placed in-line to shut off normal sewage flow that was then diverted into the onsite sump through a Y connection. The Y connection has a gate valve and an elbow attached to it to control and direct flow when sampling was being conducted. A submersible sewage grinder lift pump (Goulds RGS 2012, Goulds Pumps, Inc.) was placed in the sump and connected to the above-ground sampling tanks through a hose of 1.5 inches in diameter. The grinder pump is capable of pumping up to 41 GPM at 95 feet of head. The pump activity was controlled by a float switch and recorded by a data logger (Station-Analyzer Model 302, Marsh-McBirney, Inc.). The average flow rate was determined using the sump/pump setup in conjunction with the data logger and a current clamp. Two 30-gallon high-density polyethylene tanks were used above ground for the sampling events so that all of the flow can be captured during high-flow periods. After sewage was collected in one of the tanks it was mixed to a homogeneous solution before two samples were taken. The volume collected for each sample was about 1 liter as recommended by Benefield et al. (1980). All of the samples were preserved and stored onsite following the procedures listed in the Standard Methods (1992). After a tank's contents were sampled the sewage was returned back to the system by gravity. The elevation difference of the tanks and sewer main provided sufficient gravity for drainage. During days when influent sampling was not conducted, the sewage flow entered directly into the onsite treatment system without passing through the sump. Dissolved oxygen (DO), pH, and temperature of the sewage were measured directly in the sump. Total suspended solids (TSS) and chemical oxygen demand (COD) of the samples collected were measured in accordance with the methods listed in the Standard Methods (1992). NH_3-N concentrations of the samples were measured using a gas-sensing ammonia probe with an air gap assembly (Hach Company). Total Kjeldahl nitrogen (TKN) of the samples was measured using the Digesdahl Digestion Apparatus (Hach Company).

The hourly flow volumes of each site and their associated constituent concentrations are shown in Table 6.1. This data illustrated the highly variable conditions that can be encountered in onsite wastewater treatment systems. The data indicated that a bi-model flow pattern was observed at Site's I and II during the weekdays, with peak flows occurring from 6 am to 8 am and from 6 pm to 9 pm. Site III had no steady flow pattern observed during the sampling period possibly because one resident was at home all day. The average wastewater flow produced by the three sites as shown in Table 6.1, ranged from 16 to 38 gallons/capita/day which was somewhat lower than the average value of 44 gallons/capita/day for residential areas reported by the US EPA (1980). The 1980 study does report flow values that overlap the values found in this study. In addition data developed in the 1980 investigation included houses with plumbing that did not include low-flow, plumbing fixtures.

Table 6.1 The Measured Daily Flow Volumes of Sites I-III.

	Site I	Site II	Site III
Number of people	two working adults one infant	two working adults three children	one working adult one non-working adult
Daily Wastewater Flow Volume (gallons)			
weekday 1	64	42.3	36.4
weekday 2	46.5	139.7	30.6
weekday 3	36.4	59.3	
weekend	75	72.7	158.3
average	**55.5**	**78.5**	**75.1**

The constituent concentrations fluctuate significantly within the same day as well as day by day for all of the sites. The average daily concentrations of NH_3-N, TKN, TSS, and COD over the sampling period were 6~15 mg/L, 48~75 mg/L, 245~295 mg/L, and 623~760 mg/L, respectively, which are all within the typical ranges of residential sewage (US EPA, 1980). No synchronous cyclic patterns, which are typically observed for the influent of a municipal wastewater treatment plant, existed between flow and constituent concentrations. Additional data suggested that TSS constituted a significant portion of the measured COD and TKN. This indicated that obtaining representative samples for TSS analysis is essential not only to the correct estimation of TSS mass loading, but also to the mass loading of COD and TKN. The daily and average mass loadings (per capita) of NH_3-N, TKN, TSS, and COD at the three sites are shown in Table 6.2. The individual mass of the pollutant was estimated from its concentration in each sample and the associated flow volume recorded. The average mass loading (per capita) of the constituents analyzed at the three sites were mostly below the typical ranges reported (US EPA, 1980).

Table 6.2 The Daily and Average Mass Loadings (per capita) of Different Constituents.

Mass loading (g/capita/day)	Site I	Site II	Site III
NH_3-N	0.6 - 1.0 (0.7)*	0.1 - 0.8 (0.4)	0.9 - 4.2 (2.2)
TKN	6.3 - 8.1 (7.5)	0.8 - 7.5 (45)	3.2 - 9.3 (6.9)
TSS	17.7 - 42.1 (26.3)	4.7 - 29 (17.4)	16 - 45.4 (35)
COD	55.1 - 103.7 (79.9)	15.8 - 70.4 (42.3)	37.6 - 121.1 (88.5)

*Numbers in the parentheses are the average mass loadings per capita over the sampling period

The study outlined three possible sampling plans and discussed their advantages and disadvantages. This data and the information collected during Phase I, provided valuable information regarding the development of the formal final sampling plan used in the Phase II study.

Chapter 7 - System Selection

7.1 System Selection Criteria from Phase I

Criterion for selection of technologies was developed through discussion sessions with the team investigators, regulators, and Bernalillo County staff. The criteria employed in evaluating the alternative onsite wastewater treatment systems were:

- Units commercially available,
- Design flow of 350 – 500 gpd adequate for a single-family residence,
- Produce effluent having the potential to meet the following performance standards:
    - Total nitrogen (as N) = 10 mg/L
    - BOD_5 = 10 mg/L
    - TSS = 10 mg/L
    - Fecal coliform =100-1,000 cfu/100mL
    - pH between 6 and 9
- Unit capital cost less than $7,000,
- Low operation and maintenance costs; and
- Easy system to operate and maintain.

Commercial availability of the treatment system was chosen as one of the selection criteria because existing purchasing and maintenance channels must be available for homeowners. The system design flow of 350 to 500 gpd was chosen because it is the typical amount of wastewater generated daily from a three- or four-bedroom home. The performance standards are used as a goal to select potential onsite systems. For the most part all criteria were given equal rating, but the units had to show some evidence of nitrogen removal capabilities or they were not considered. Some subjective analysis was employed especially when vendor information was vague. This lack of specifics had to be tolerated in the selection process because many companies have what they consider as proprietary concerns that must be respected.

Information about alternative onsite wastewater treatment systems was collected through literature reviews, discussions with the county and city professional staff, and by directly contacting vendors and manufacturers. Geary (1988) produced the following general list of alternative systems or modifications considered suitable for onsite treatment to improve effluent discharges:

- Shallower placement of distribution network,
- Replacing soil with another soil of improved characteristics,
- Alternating drain fields,
- Mound systems,
- Aquaculture,
- Evapotranspiration/absorption,
- Sand filters,
- Aerobic treatment,
- Split systems for black and greywaters,
- Non-water carriage toilets, and
- Low water-use toilets.

Despite this impressive list of decentralized treatment systems, these investigators still felt that strong centralized management was necessary to assure proper installation, operation, and maintenance. In addition, this list did not address nitrogen removal as a specific criterion.

Whitmyer et al., (1991) ranked various onsite treatment systems which included nitrogen removal as a criterion. The results indicated that various configurations of sand filters were ranked first followed closely by septic tank/peat filters (modified soil), and the RUCK system. This same survey concluded that technology for nitrogen removal applied to onsite systems is relatively untested and very little data exist regarding removal, consistency, and reliability. In addition, almost no data exist relating nitrogen removal benefits to total system costs. While the survey did include some information on constructed wetlands, many systems were not included because data was not available.

Recent information provided by vendors has indicated an increased interest in nitrogen removal and that more data is now available. Many manufacturers expressed interest in the project and were willing to contribute information about their systems. While a number of experimental systems were previously identified in a review by Zachritz (1994), only commercially available units were included for the present analysis. This decision was made to facilitate the implementation of systems for onsite application and to provide motivation for vendors to modify their existing systems to remove nitrogen.

Seventeen onsite treatment systems were evaluated using the criteria outlined above for their feasibility to be implemented in Albuquerque and Bernalillo County. These onsite systems are (in an alphabetical order):

- Clearstream
- Clearwater
- Clean Water Ozone System
- Constructed Wetland (subsurface flow system)
- Fluidyne
- Greenroom/Rockmarsh
- Hydro-Action
- Hydroxyl Systems
- Jet Inc.
- Krofta Compact Clarifier
- Micronair
- Multiflo
- Recirculating Trickling Filter (RTF)
- Scienco/FAST
- Smart Separation
- Stahlermatic
- Whitewater

All of which are registered trademarks of their respective systems.

Based on our analysis using the fore-mentioned technology selection criteria, five of the 17 potential alternative onsite wastewater treatment systems were tentatively selected for field demonstration at selected sites in Bernalillo County. Septic tanks, and new and mature constructed wetlands, were added to the list for several reasons. Septic tanks are the approved method of treatment for most of New Mexico and the U.S. and, therefore, represent a baseline standard of performance. The new and mature constructed wetlands system were also included as test systems because many wetland systems are already installed in the study area, and many regulators are interested in the performance of these systems. The other technologies evaluated, were prioritized and ranked based on how well they met the technology selection criteria.

The following was the preliminary list of the conventional and alternative technologies that were selected as final candidates for inclusion in the field demonstration phase of the study:

- Septic Tank
- New Constructed Wetlands
- Mature Constructed Wetlands
- Clearstream
- Scienco/FAST
- Whitewater
- Hydroxyl Systems
- Fluidyne Inc.

7.2 Site Selection Criteria

The general site selection criteria were developed as guidelines for selecting test sites. These sites could be in any location within Bernalillo County, but the South Valley and the East Mountain area were of particular interest. Demonstration sites for the alternative onsite treatment systems were selected based on the following criteria:

- Not in a sewered area and must require onsite treatment system;
- Three- to four-bedroom domestic dwelling producing typical wastewater;
- Typical household activities avoiding sites with small business operations or hobbies such as small family-operated restaurants, photo developing, or jewelry making;
- Enough space to install the demonstration system;
- Owners agree to participate in the demonstration project by providing $1,500 toward purchase/installation of the demonstration system; and
- Provide a disposal system for the treated wastewater
- Provide electricity and/or telephone connection to the system if required
- Perform home owner's regular operation/maintenance
- Provide access to the system during samplings
- Notify about major changes in wastewater flow (vacation, additional people)
- Sign a fully implemented home owners agreement

The final selection and implementation of a specific technology depended on finding suitable sites that could meet most of the criteria outlined above. For liability reasons, the homeowners had to agree to sign a contractual agreement outlining the costs to be paid to them and the time frame of the study.

7.3 Final Systems Selected

Final system selection depended on three major factors: finding a suitable site, agreement of the homeowner with the conditions of the study, and final agreements with the manufacturer of the unit for installation. Finding suitable sites was problematic because of the complex nature of matching system requirements to site requirements and getting the home homeowner to agree to participate. Some of the selected technologies did not have manufacturer representatives in the area. The installers were not familiar with the technology and had very little motivation to do the installation. Many installers had jobs lined up well in advance and had waiting lists. Getting them to do our installations was not always easy. It was custom work requiring extra time that was not always compensated for proportionally to the time spent. The sumps were not typical and many installers did not want to be responsible for a possible “failing system”. Because much of the installation was considered custom for the sample sumps, one member of the research team had to be onsite to insure the system was installed properly. This created more delays and required greater coordination. Installers can be very independent and some jobs require more time than others.

Homeowners were problematic as well. Some did not want to agree to the conditions of the homeowner’s agreements. One owner backed out of the second phase study after participating in the first phase. Owners wanted to be paid up front while the research team felt they should be paid on a monthly basis. Some homeowners threatened to withdraw from the project if they were not paid up front. We paid them upfront. The timing of construction and occupancy of the house was also a complication. We tried to find houses that were under construction or needed retrofit repairs. The house needed to be built, the system installed, and an occupant in the house within three to six months or the sample period requirements would be invalidated. One selected site became invalid when the owners filed for divorce and changed the occupancy from four to one occupants and the house was put on the market.

Sites that qualified for the study were not easy to find. Some sites were just being developed and had a long lead-time to construction. Some did not have the right number of bedrooms or were multifamily dwellings. Some did not require an alternative system and the owners were not willing to spend more for a system they did not need and we could not afford to offer more than the budgeted amount. Two very promising systems the Hydroxyl and Fluidyne Systems, were dropped from the study. Both offered interesting technologies and looked promising to meet nitrogen standards. Hydroxyl had a site and had indicated that the budgeted amount could be met and they were willing to participate and the homeowner was willing to sign the agreement. However, within a month of the projected installation date, the company withdrew because of a corporate decision to stop selling to the single family onsite market. Fluidyne did not have a local representative and this made finding a potential site difficult. Several sites were located, but did not work out because of some of the reasons previously mentioned. One site that later became the Whitewater test site, could not be used because the space requirements by the Fluidyne system was somewhat larger than the Whitewater system. The proposed site was space-limited by set back requirements and easements, so the Fluidyne system could not be physically located on the site. We think this system deserves consideration and should be tested in the future.

The second treatment wetlands option was also dropped. This would have included a newer system versus an older system and could have included one of the more advanced wetland designs that provides aeration. An older designed wetland system with a mature plant bed was selected. The research team felt that the newer wetland designs should also be considered since this represents designs that are specific for enhanced nitrogen removal.

Several systems such as slow sand filters with Orenco technology and Bioclere systems were considered, but the reported unit cost for these systems was above $10,000. This cost was above the acceptable cost ceiling of $7,000. These systems might meet the proposed treatment standards and may be considered in the future. The following is the list of the conventional and alternative technologies that were field demonstrated for this study:

- Septic Tank
- Mature Constructed Wetlands
- Clearstream
- FAST
- Whitewater

Generally, these systems were installed or the existing system altered to accept the sample sumps. Subcontracts were developed with the installers to pay a portion of the work directly or reimburse the homeowner. Homeowner's agreements (see Appendix F for a sample) were developed and signed by NMSU and the occupant. These were finalized and issued and reimbursements then had to be processed and issued to the various parties. The sample sumps were installed, tested, and sampling began. Over the course of the project, costs for fixing or replacing various items were paid by the research team. In one instance and emergency power problem, that was taken care of by the homeowner was also paid for by the project.

Chapter 8 - Test Methods

Five onsite wastewater treatment systems were evaluated for this study. The systems were installed by local licensed installers at various locations in Bernalillo County, New Mexico. The systems were installed at typical three bedroom residences and were owner operated with no assistance from the research team. All sites were modified to allow sampling of system influent and effluent wastewater and measurement of flowrates. The basic sampling layout is shown in Figure 8.1. The objective of the sample plan was to evaluate the selected wastewater treatment systems at actual home sites in various locales within the Bernalillo County area. The project goal was to work with typical installed onsite systems; not optimized systems at a controlled site. In addition, the sites were sampled over an eight to fourteen-month period to measure the effect of seasonal temperature and usage. Wastewater samples were taken from the influent and effluent of each system twice per month. All systems in the study were sampled in a similar manner, deviating only to accommodate plumbing or system configurations as necessary. The influent samples were collected as composite grab samples where discrete system flows were allowed to accumulate overnight in the sump and then mixed and sampled. The effluent samples were collected as composite grab samples where a portion of the discrete system flows were allowed to accumulate and mix in a collection bucket. They were then preserved as required by the testing laboratory.

This sampling regime allowed for the statistical comparison of the influent and effluent of each system and the performance of each system. Installation of sample sumps and systems was conducted by licensed contractors and all electrical connections were performed by licensed electrical contractors.

8.1 Influent Sampling

To obtain influent samples, the sample basin (Figure 8.2), which operates in a by-pass mode most of the month, was activated by closing valves and allowing the sumps to collect influent wastewater. This was initiated in the afternoon thereby beginning the sampling period. The basin was allowed to fill and overflow to collect a representative sample covering the period from about 4:00 pm till 9:00 am the next morning. After collection in the sump, the sample was influent sample was then collected by dipper from the sump. The existing plumbing on one of

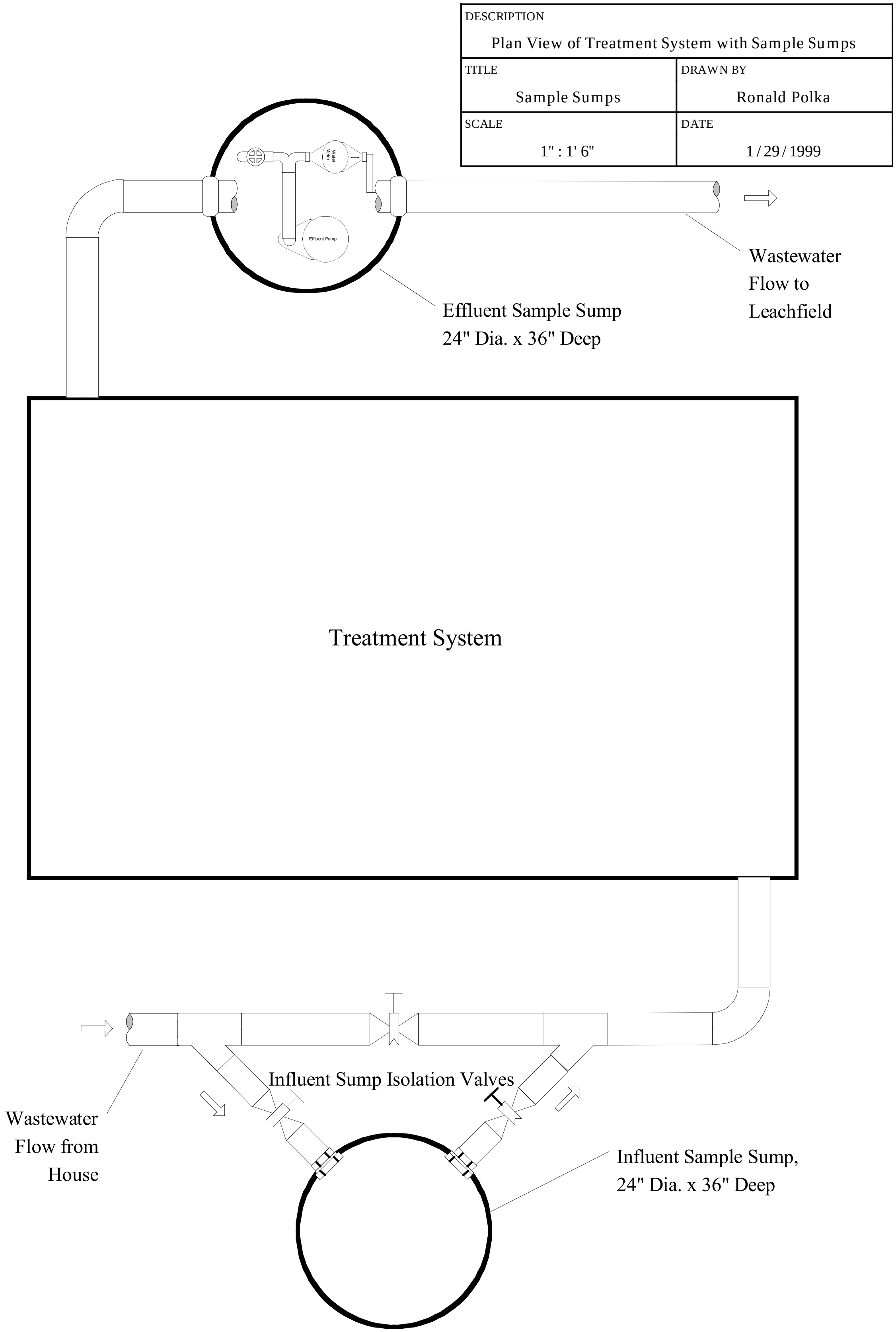

Figure 8.1. General Layout of all Test Systems with Influent and Effluent Sample Sumps.

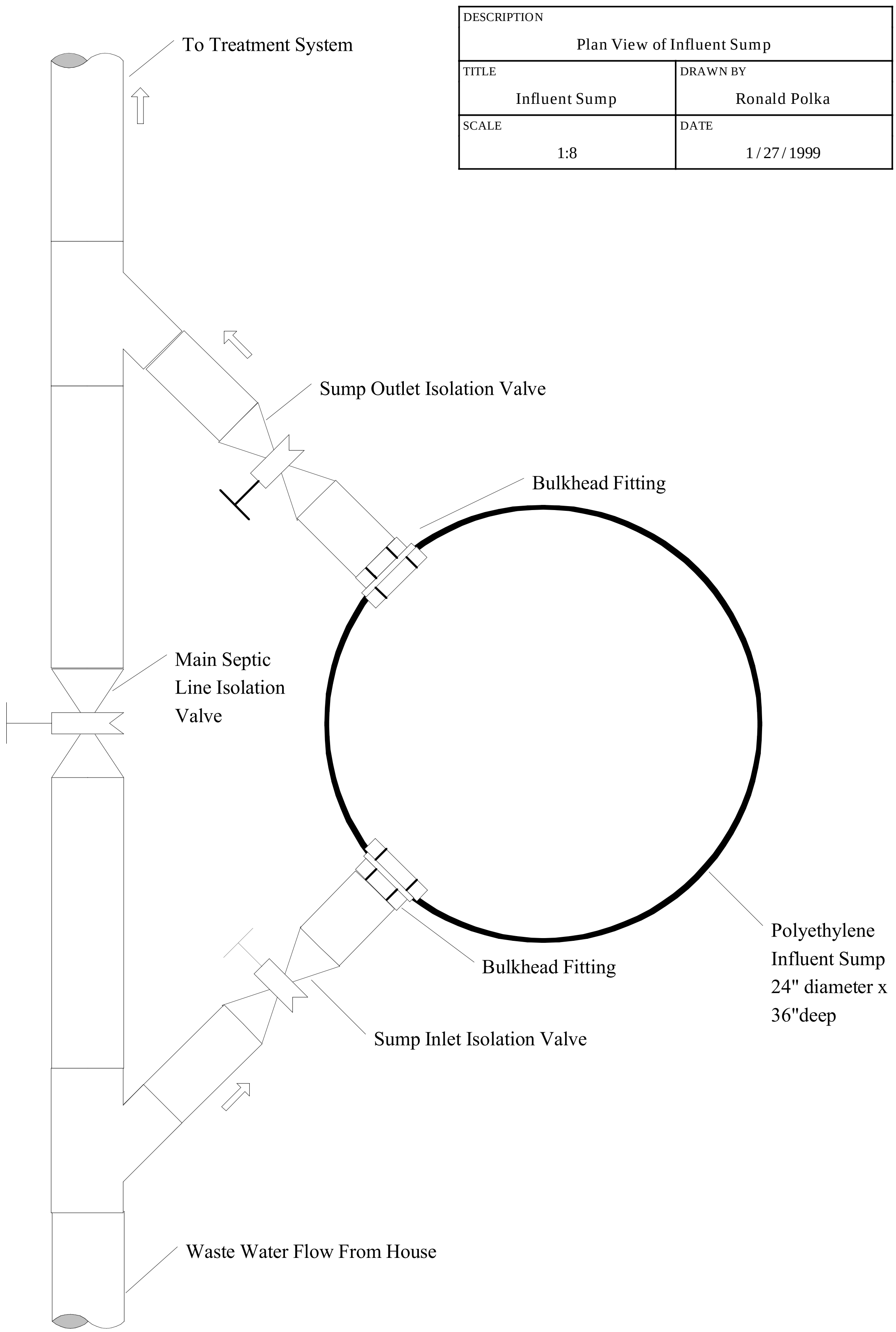

Figure 8.2. Typical Layout of Influent Sample Sump.

homogenized using a submersible sewage pump (Simer Model No. 2960ss). A simple grab the tested systems (the Constructed Wetland) necessitated the use of a one gallon mixing bucket rather than a large influent sump.

Samples from all the sites were collected, iced to 4 degrees, stored in an ice chest, then transported to the laboratory for analysis. Two 1,000-ml samples and one 100-ml sample were taken from the homogenized wastewater. One sample was preserved with H_2SO4 (for COD and TKN) the other sample was preserved by chilling in an ice water bath (for TSS). The 100-ml sample was preserved with thiosulfate (for fecal coliform). Measurements of temperature, pH, conductivity, and dissolved oxygen were taken in the field on each collected sample. Temperatures were measured with a partial immersion bimetal dial thermometer. pH was measured with an Orion ® model 290A Portable pH/ISE Meter. Electrical conductivity was measured with a YSI model 33 Salinity/Conductivity/Temperature meter. Dissolved oxygen was measured with a YSI model 51B Dissolved Oxygen Meter. All field-recorded data was entered into a field notebook. Field Data and laboratory results were compiled into an Excel database for review and analysis. Table 8.1 shows the sampling frequency preservation techniques container's uses and holding times for different analytes.

All laboratory result forms were kept on file as a backup and a field notebook was kept by each team. In addition, a formal chain of custody and sample numbering system was used as specified by the City of Albuquerque Water Quality Laboratory. Samples were submitted to the Laboratory using a Water Quality Laboratory (WQL) General Use Sample Submission Form. This form was time/date stamped at the Laboratory and filled out with the test program and protocol identifications, sample point ID, address, and recorded field data. The submitter of the samples was required to initial and sign the submission form. The receiving clerk also signed the form upon completion. During checking at the Laboratory each sample (consisting of 3 sample bottles) received a unique WQL Sample Number. All samples delivered as a lot (influent and effluent samples from one or more systems) were assigned a single LIMS Submission ID Number. All tests were performed in accordance with Standard Methods (APHA 1996). Additionally, observations regarding general operation of the system, any reported problems from the owner, odors in the general vicinity of the system and sumps, excess water or flooding,

Table 8.1 Sampling Frequency, Containers, Preservation Techniques, and Holding Times.

Analyte	Sampling Frequency	Container	Preservative	Maximum Holding Time
Temperature	2 /mo	In situ	NA	Analyze immed.
Dissolved oxygen	2 / mo	In situ	NA	Analyze immed.
pH	2 / mo	In situ	NA	Analyze immed.
Electrical Conductivity μmhos	2 / mo	In situ	NA	Analyze immed.
Ammonia, Total	2 / mo	Subsampled from 1000 mL HDPE bottle #1	H_2SO_4, pH < 2	14 days
Biological Oxygen Demand, 5-d	2 / mo	Subsampled from 1000 mL HDPE bottle #2	<4°C	7 days
Bromide, Dissolved	2 / mo	Subsampled from 1000 mL HDPE bottle #2	<4°C	14 days
Chloride, Dissolved	2 / mo	Subsampled from 1000 mL HDPE bottle #2	<4°C	14 days
Chemical Oxygen Demand	2 / mo	Subsampled from 1000 mL HDPE bottle #2	<4°C	14 days
Nitrate & Nitrite	2 / mo	Subsampled from 1000 mL HDPE bottle #1	H_2SO_4, pH < 2	14 days
Orthophosphorous	2 / mo	Subsampled from 1000 mL HDPE bottle #2	<4°C	14 days
Sulfate	2 / mo	Subsampled from 1000 mL HDPE bottle #2	<4°C	14 days
Total Kjeldahl Nitrogen	2 / mo	Subsampled from 1000 mL HDPE bottle #1	H_2SO_4, pH < 2	14 days
TSS & DSS	2 / mo	Subsampled from 1000 mL HDPE bottle #2	4°C	3 days
Fecal Coliform	2 / mo	Subsampled from 100 mL HDPE bottle #3	Sterilized container, Thiosulfate	5 days

plugging etc. were noted in the field notebook. Also conditions such as frozen soil, snow cover, general climate, or noises from the unit were all important qualitative factors that were noted when necessary. These notes were kept in the field logbooks and photocopied at intervals throughout the study.

8.2 Influent Characterization

Characterization of influent flows by monitoring system flow events was conducted for the following systems, Whitewater, Scienco/FAST, and the Constructed Wetland. These data were recorded during the automated tracer analysis tests. The influent flows into the systems were monitored by recording pump events at the effluent sump. Pump cycles were automatically recorded on a data acquisition system by monitoring AC power usage. This provided pump on and off times plus the duration of each cycle. This enabled a determination of flow frequency and size over the duration of the study period, two to four weeks depending on the system. Pump flowrates were considered constant over the test period because the system head that the pump saw did not vary. Flowrates were determined by simply dividing the total pump operating time by the recorded water use from the effluent flowmeter. Flow event volumes and durations were then calculated based on pump operating times.

8.3 Effluent Sampling

The sample basin (Figure 8.3) was operated in a flow-through mode for most of the month. In the afternoon at the beginning of the sample period the water meter reading was recorded to provide a sample period start value. Sample collection was activated by opening the sample valve and placing a one-gallon container under the open sample valve. From this time until the end of the sampling period the container received a small flow from the pumped side of the effluent waste stream. This container was set up to receive a sample at the same time as the influent sump. The container would fill and overflow during the sample period and contain a representative sample covering the period from about 4:00 pm until 9:00 am the next morning.

Two 1,000-ml samples and one 100-ml sample were taken from the composite one gallon sample of system effluent wastewater. One sample was preserved with H_2SO4 (for COD and TKN) the other sample was preserved by chilling in an ice water bath (for TSS). The 100-ml

DESCRIPTION	
Elevation View of Effluent Sump	
TITLE	DRAWN BY
Effluent Sump	Ronald Polka
SCALE	DATE
3 / 16 : 1	1 / 27 / 1999

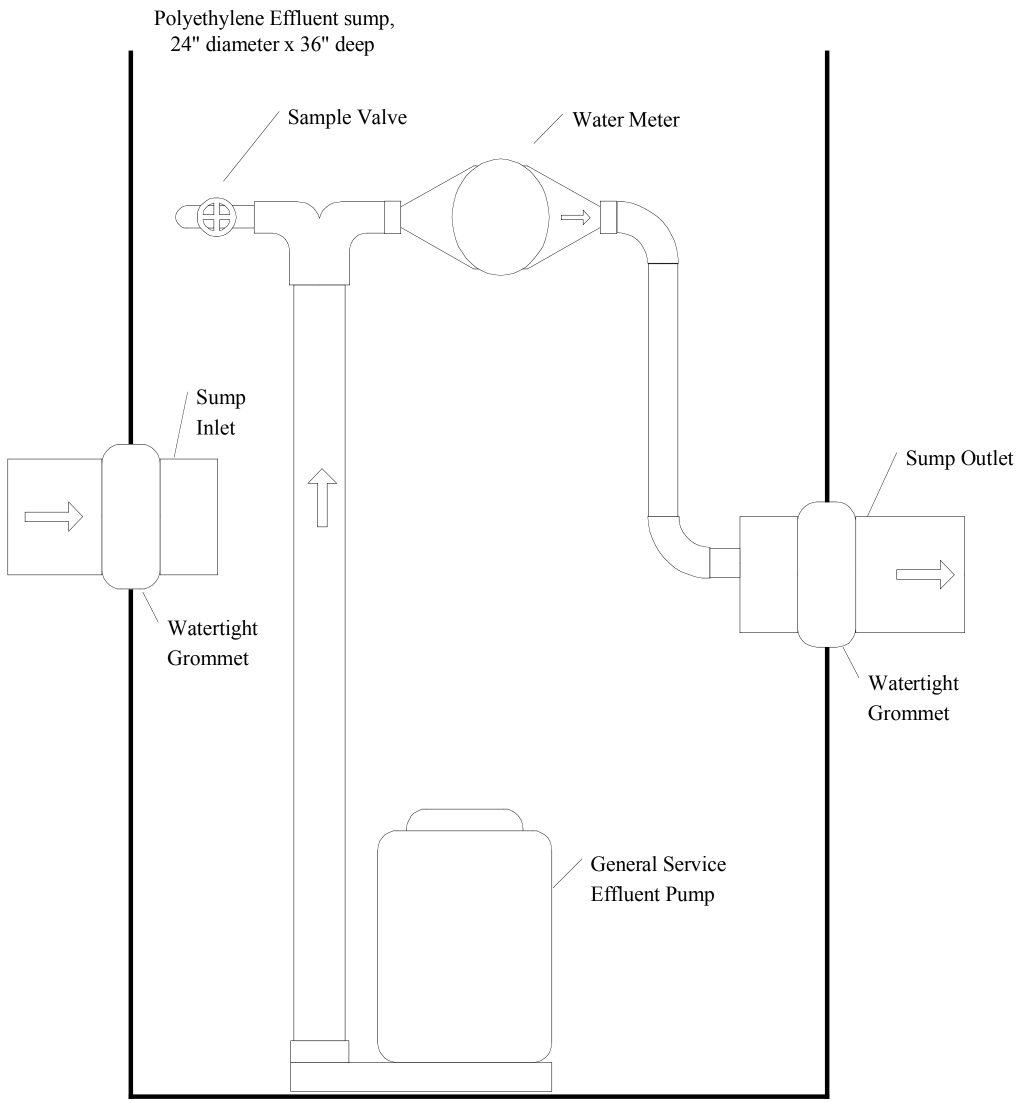

Figure 8.3. Typical Layout of Effluent Sample Sump.

sample was preserved with thiosulfate (for fecal coliform). Measurements of temperature, pH, conductivity, and dissolved oxygen were taken in the field on each collected sample. Samples were stored in an ice chest, and transported to the laboratory for analysis.

8.4 System Flowrates

The effluent flowrates for all systems were measured using positive displacement water meters (Neptune or Badger) that provide an accumulated flow in gallons (Figure 8.3). These meters were connected in a way to provide quick disconnect for servicing and adjustment. The flow meters were read at approximately the same intervals of sampling to determine an accumulated flow since the last sample period and an overnight flow for the actual period of sample collection. The accumulated flow was reported as an averaged daily flow for the site, and compared to the over night flow. A typical meter arrangement in the effluent sample sump is detailed in Figure 8.3.

Accumulated flow was recorded and compared to previous flow information. This data was evaluated onsite to assure that the flow meter was operational. In addition, the meter would be checked from the period covering the beginning of sampling (day 1) to the period ending the next day to be assured that the meter was operating properly. During the course of the project two incidents of meter failure were observed due to jamming by foreign objects. These problems were readily solved by meter replacement at the site and subsequent maintenance and cleaning of the affected meter. The new meter readings or corrected readings were recorded to assure accurate information for the next trip.

8.5 Reactor Analysis

In order to verify the reactor hydraulics and system detention times, tracer analyses of three systems were conducted. A bromide tracer was introduced into a cleanout port of the septic line between the influent sump and the system and was used to characterize the reactor hydraulics of the tested system. The amount of tracer used depended on the estimated volume of the system. The NaBr solution was mixed to a concentration of 100,000mg Br-/Liter H_2O and introduced into the system in a single pulse. This pulse was then followed by a 5-gallon pulse of tap water to flush the Bromide tracer down the pipe and into the system.

A Campbell Scientific CR500 portable Data Acquisition System (DAS), was used for collecting the following data. The Bromide concentration was measured with a Cole Palmer ® Bromide Electrode model 27502-04. The effluent water temperature was measured with a Campbell Scientific Temperature Probe model 107. The DAS monitored the status of the system at one-second intervals during the course of the tracer study. Effluent sump pump operation was monitored through the use of an AC/DC converter. During pump off periods the DAS remained in a quiescent state, waking up and recording data only during pump on cycles. The system flowrate was determined by monitoring the on and off times of the system pump. From this data pump operating times, pump cycle duration, and an average pump flowrate was calculated. The Bromide probe was arranged using a mixing cup procedure (Levenspiel, 1989) where flow from the effluent of the unit was allowed to flow by gravity into the sump. As the sump received flow from the system it mixed to form a homogeneous sample. During pump operation, the Bromide concentration was measured at one-second intervals and averaged over the pump cycle to determine a concentration for that particular pump cycle. The data was monitored and stored by the DAS unit. Run times of the pump were also monitored and recorded. Typically, the DAS unit was field deployed for about 2 weeks for data collection. The one exception was the Constructed Wetlands that were monitored for 4 weeks. The raw data was downloaded to an Excel spreadsheet for analysis. The tracer analysis raw data was analyzed for detention time and flow patterns, using methods outlined by Levenspiel (1993). Next, the raw data was inputted into the Impulse software (Baddock and Brouckaert 1992) to model the system and thus, better define its flow pattern. An average flowrate (total volume pumped during study/length in hours of tracer study), was used as input into the Impulse program along with the total mass of tracer added in milligrams (mg) of Bromide.

Tracer studies were conducted for the Whitewater, Scienco/FAST, and the Constructed Wetlands systems as pulse experiments. The raw data was in the form of tracer (bromide) concentration in milligrams per liter (mg/L) in the fluid leaving the reactor vessel versus (vs.) time in hours, which is referred to as the response curve or C vs. t curve. The C vs. t data was used to evaluate the mean and variance of the tracer curve. The mean and variance of a tracer curve are the two most useful measures used in all areas of tracer experimentation. Levenspiel (1993) describes the mean as follows: the mean tells when a curve passes the measuring point, locates its center of gravity in time. The mean is also referred to as the mean detention time of the system. The variance tells how spread out in time, or how "fat" the curve is. To evaluate the mean and variance from pulse experiments, the following equations need to be solved:

The area under the curve: $= \int Cdt$

The mean: $= \int tCdt / \int Cdt$

The variance: $= (\int t^2 Cdt / \int Cdt) - \text{mean}^2$

Where, t = time, hours

C = tracer concentration, mg/L

However, if the data points are numerous and closely spaced the mean and variance can be estimated by the following:

The mean: $= \sum t_i C_i \Delta t_i / \sum C_i \Delta t_i$, (8.1)

The variance: $= (\sum t_i^2 C_i \Delta t_i / \sum C_i \Delta t_i) - \text{mean}^2$. (8.2)

In order to use equations 8.1 and 8.2 for determining the mean and variance, linear interpolation was conducted for the C vs. t data. Since the data was collected by instantaneous readings, then for a set of n C vs. t readings the mean and variance were calculated by:

Mean $= \sum (t_{i+1} + t_i)(C_{i+1} + C_I)(t_{i+1} - t_i) / 2\sum (C_{i+1} + C_i)(t_{i+1} - t_i)$ (8.3)

Variance $= (\sum (t_i + t_{i+1})^2 (C_i + C_{i+1})(t_{i+1} - t_i) / 4\sum (C_i + C_{i+1})(t_{i+1} - t_i)) - \text{mean}^2$ (8.4)

Once the mean and variance were calculated for each pulse experiment, an E_t curve was constructed using the C vs. t data. The E_t curve is also called the exit age distribution function or response time distribution (RTD) function, which represents the time spent in the vessel by the flowing material. The E_t curve was then converted to its dimensionless time scale form (E_θ) and used for analysis of reactor vessel behavior. To convert the C vs. t curve into the E_t curve, the C (concentration), values were divided by the total area under the C vs. t curve, while the time (t) values remain the same. The total area was estimated by the following:

Total area $= \sum C_i \Delta t_i$ (8.5)

Next, to convert the E_t curve to its dimensionless form E_θ, the E_t curve C/area values were multiplied by the calculated tracer curve mean. The time (t) values were then divided by the mean to give a dimensionless time scale θ. The E_θ dimensionless curve will be referred to as the E curve for the remainder of this report. The area under the E curve is unity. The C vs. t curves and the E curves for the Whitewater, FAST and the Constructed Wetlands systems are presented in the "Hydraulic Analysis" section for each system. The raw data and calculations are presented in the Appendix. All calculations were conducted using Microsoft Excel.

As mentioned previously, the E curve was used as a tool for analysis of the reactor vessel, namely in terms of determining the flow pattern in the vessel. The E curves for the pulse experiments pertaining to the three systems mentioned above were compared to typical E curves for ideal plug flow, mixed flow and the intermediate flow pattern of plug flow with dispersion. The comparison was purely based on the general shape of the actual E curve. A determination was made whether the system seemed to exhibit plug flow, mixed flow or plug flow with dispersion. Once this determination was made, the system was simulated using the Impulse program to perform a regression on the actual C vs. t data against a theoretical curve of the selected model (plug flow, mixed flow or plug flow with dispersion).

Impulse is a DOS based program that enables a network of ideal components (complete mixed flow reactors, plug flow reactors, plug flow reactors with dispersion, and a non integral number of complete mixed flow reactors in a series), to be connected by input blocks, flow mixers/splitters, recycle loops, and output blocks to simulate a real flow system. The input data to the program are measured (experimental) concentration and flow values as a function of time. The outputs from the program are theoretical concentration and flow values (after any vessel) as a function of time. The program can be used to regress for any unknown parameter or parameters provided an experimental output curve (tracer curve) is provided. The Impulse program was downloaded from the internet at: http://www.und.ac.za/und/prg/impulse/impulse.html. The user manual was also downloaded from this website.

For the purposes of this study, the inputs to the Impulse program were the tracer curve (C vs. t data), average flowrate into the system and the actual mass of Bromide tracer injected into the system. The program was then run in the regression mode, where the program output yielded a theoretical C vs. t curve and a calculated reactor vessel volume. The theoretical (simulated) C vs. t curve was plotted on the same graph as the actual C vs. t curve. Visual comparison of the two curves along with knowledge of the internal details of each system and basic reactor design criteria were then used to determine whether the theoretical model chosen was representative of the actual system. The Impulse program output was downloaded to Microsoft Excel spreadsheets and used for further system analysis. The theoretical C vs. t curves developed by using the C vs. t data output generated by Impulse for each of the three systems previously mentioned are found in the “Hydraulic Analysis” section for each system. The C vs. t data from which these curves were constructed is found in the Appendix.

For the Whitewater and FAST systems, two regression scenarios each for the plug flow (PF) with dispersion and complete mix flow (CMF) models, a total of four altogether were run on Impulse. Namely, for the PF with dispersion model, one scenario consisted of varying the flowrate to obtain the actual mass of Bromide tracer into the system and using the actual C vs. t data as an input to the Impulse program. The second scenario for the PF with dispersion model consisted of varying the reactor volume to obtain the actual mass of Bromide tracer into the system and using the actual C vs. t data as an input to the Impulse program. These two scenarios were then run using the CMF model making up a total of four. The output produced by the Impulse program consisted of theoretical C vs. t data for each scenario of the PF with dispersion model and the CMF model, as well as calculated flowrates and volumes. The Impulse results from the four simulated scenarios were compared to the actual system on the basis of actual or measured bromide recovery and the reactor vessel volume. The Impulse program results were then scrutinized based on the program limitations detailed in the Impulse user manual. The limitations of interest used for this study are the following:

- The experimental determination of the tracer response (C vs. t) curve is not trivial and may influence the result. Thus a user must be aware of the techniques for tracer tests, and be able to estimate experimental error and how it influences the curve;

- Impulse should only be used to model continuous or near continuous flow systems with constant flowrate.

Although the systems modeled were not at constant flowrate, the tracer study data was collected over a long enough period of time to allow for the use of average flowrates rather than instantaneous actual flowrate readings. In addition, the samples taken were composite samples, which consisted of several flowrate events averaged together.

8.6 Statistical Analysis

Statistical analysis of the water quality data from each system was performed using statistical packages in Excel Version 7.0 and Sigma Plot Version 2.0. Means for each influent and effluent were calculated and standard error, standard deviations, median, maximum and minimum values were also determined. A one way analysis of variance (ANOVA) technique was used to compare influent and effluent values for each system. This assumes a condition where the sample means are equal and tests this hypothesis at a confidence level of 95 percent and calculates a F statistic and a p-value for the condition of the test. A p-value less than 0.05 rejects the hypothesis that the means are equal. A p-value greater than 0.05 fails to reject the hypothesis and the means are considered statistically equal or there is no significant difference in the means. The lower or higher the p-value, the greater the prevailing condition of equal or not equal is supported by the statistics. A small sample number or a large standard deviation (variation in the data) can affect the p-value. These conditions were noted where obvious differences in calculated mean values were encountered, but the difference was not supported by the calculated statistics.

Chapter 9 - System Evaluation Criteria

In order to accomplish the goals and objectives of this research a multi-faceted approach was employed. The tools used in reaching the goals included: field-testing selected technologies, reviewing pertinent literature, reviewing manufacturer data, surveying installer and homeowners, interviews with regulatory personnel, attendance of conferences and workshops, and in-depth reviews of other state's existing or proposed regulations. This data was synthesized to form the basis for the recommendations developed in this report.

9.1 Installation

The evaluation criteria used for system installation, was a combination of several factors. First, from manufacturer supplied data the methods of installation were reviewed for each system. This information was examined for internal consistency, engineered reliability, and practicality. Where possible, installation of the system selected for study was observed. Field notes were taken and attitudes of both the installer and the manufacturer representative were noted. The amount of technical information provided by the installer was observed and the degree of actual in the field onsite training was noted. The length of time and the timing of the manufacturer visits to the site during installation were noted. This was some what biased because the manufacturer knew in advance that we would be at the site and the implications of our presence.

The research team also observed the condition of the onsite systems that had already been installed before the beginning of the study. Such aspects as the quality of the work in terms of plumbing and electrical, the general layout of the units and the individual components, settling of the treatment units, the quality of the finish work (the above ground access ports and the backfill around the unit) was noted. Additional information was noted from interviews with regulatory personnel, installers, researchers, literature data, previous reports, homeowner comments, and comments from attendees at Bernalillo County organized workshops. All of this was used to develop the guidelines pertaining to onsite system installation.

Finally, two formal surveys were conducted for installers and homeowners. These surveys were conducted by the research team via phone interviews. The installer survey used a database of over 300 installers registered in Bernalillo County. This survey was developed by the research team and reviewed by City and County personnel. Questions were asked about the numbers of installations, types of installations, use of alternative systems, typical problems, degree of training received, perceived quality of the existing regulations, and needs for improved regulations. The homeowner survey used a database of over 1,000 homeowners living in Bernalillo County and owning an onsite treatment system. This survey was developed by the research team and reviewed by City and County personnel. Questions were asked about their system, types of system, typical problems, degree of training received, cost of system, and their perception of how well the existing regulations protected groundwater and environmental health.

9.2 Maintenance

Evaluation criteria for system maintenance were developed using factors and methods similar to the installation criteria. Recommended maintenance procedures were reviewed for each system from the manufacturer supplied data. This information was examined for consistency and completeness, and practicality. All systems were observed for their maintenance needs and requirements and records were kept if maintenance by anyone (homeowner and manufacturer representative) was performed. The amount of technical information provided by the installer to homeowner was observed and the degree of onsite field training was noted. Additional information was noted from interviews with regulatory personnel, installers, researchers, literature data, previous reports, homeowner comments, and comments from attendees at Bernalillo County organized workshops. All of this was used to develop the guidelines pertaining to onsite system installation.

Two formal surveys were conducted for installers and homeowners that included questions regarding maintenance. These surveys were conducted by the research team via phone interview. Installers were asked about the amount maintenance, types and frequency, mandatory maintenance, typical problems, degree of training received, perceived quality of the existing regulations, and needs for new and improved regulations. The homeowners were questioned about maintaining their system, types of maintenance, typical problems, and degree of maintenance training received. This survey information was formally compiled and is provided as a section of the report. The other information formed the basis for the development of the guidelines.

9.3 Inspection

Evaluation criteria for system inspection were developed using factors and methods similar to the previous criteria. However this information was collected using a less direct method. Manufacturer supplied data generally did not include needs for inspection. All system installations were observed for their inspection needs. The installers were interviewed to determine their attitude towards the inspection process and the role of the inspectors. Additional information was again noted from interviews with regulatory personnel, installers, researchers, literature data, previous reports, homeowner comments, and comments from attendees at Bernalillo County organized workshops. All of this was used to develop the guidelines pertaining to onsite system inspection.

The two formal surveys also had some questions that were related to inspection issues, such as reasons for failed systems, adequacy of existing regulations and other issues. This survey information was formally compiled and is provided as a section of the report. The other information formed the basis for the development of the guidelines.

9.4 Performance

The systems were evaluated by conducting a comprehensive sampling and analysis program. This program used a “hands off” operational approach that depended on ongoing maintenance provided by the installer, manufacturer's representative, or the homeowner to operate and adjust the system. The authors only found it necessary to intervene twice. On both occasions the intervention occurred after 4+ weeks of system failure. The first part of the plan was to systematically sample the influent and effluent for each system. This was performed about every two weeks or twice per month over the course of the study period. The quantity of samples taken provided a sufficient statistical basis to evaluate the performance of each system. The use of statistics provided a sound scientific justification for the results observed. This ongoing sample regime allowed the research team to observe possible startup problems, seasonal variations, or variations in operation due to water softeners or system component failure.

Samples were collected using a modified compositing approach. A sample sump with a fixed volume was put online the day sampling began. This sump collected wastewater as it flowed through the sump over a prescribed period of time. Typically the sump was set up to collect sample from about four PM until about eight AM the following morning. The wastewater flow was allowed to fill the sump and overflow to the unit or discharge to the disposal field. The incoming wastewater mixed with wastewater from the previous event and as each event occurred, this mixing continued. Some washout loss and dilution effects from previous events were expected, but a consistent flow equalized sample was collected at the end of the sample collection period. These samples were further processed by homogenizing the sump contents with a Vortex type sewage pump, which macerated the solids and provided a uniform particle sizes. Samples were taken, preserved, and transported to the Albuquerque City Laboratory for analysis. Field analyis was conducted onsite on the sump prior to pumping. General observations regarding odors, noise, mechanical problems, and occupancy, were also noted.

Influent and effluent values for each water quality parameter results were analyzed for a suite of basic statistical parameters that included mean standard deviation, percent errors, maximum and minimum values to provide basic information about the data variability. The influent and effluent parameter means were compared using a one way analysis of variance (ANOVA). This allowed the team to determine comparative performance of water quality category for influent and effluent. This was performed for all systems. Some systems did not operate continuously throughout the study period and statistics were not valid on these systems. This is noted in the discussion. Finally the results from each system were compared to the proposed onsite treatment performance standards. Direct statistical analysis between systems with such differing influent and effluent values would have been difficult to statistically justify.

Chapter 10 - Systems

The systems considered for this study ranged from processes that included unit operations that were solely dependent on physical/chemical processes to unit operations that were a mixture of biological and physical chemical processes. The systems that were eventually field-tested consisted of mixed processes. Most systems had a sedimentation step as pretreatment to remove suspended solids and separate out floating materials such as oil and grease. This process is typically performed by a conventional septic tank. The partially treated wastewater is then treated by biological processes. Biological treatment generally consisted of anaerobic, anoxic, and aerobic processes. The sequence of these processes in the treatment train determines the degree of overall treatment performance that can be expected from a particular unit. Simple treatment systems such as the conventional septic tank (Figure 10.1) may just use anaerobic processes with some limited aerobic activity, while more complex systems may employ a mixture of biological processes in a controlled sequence.

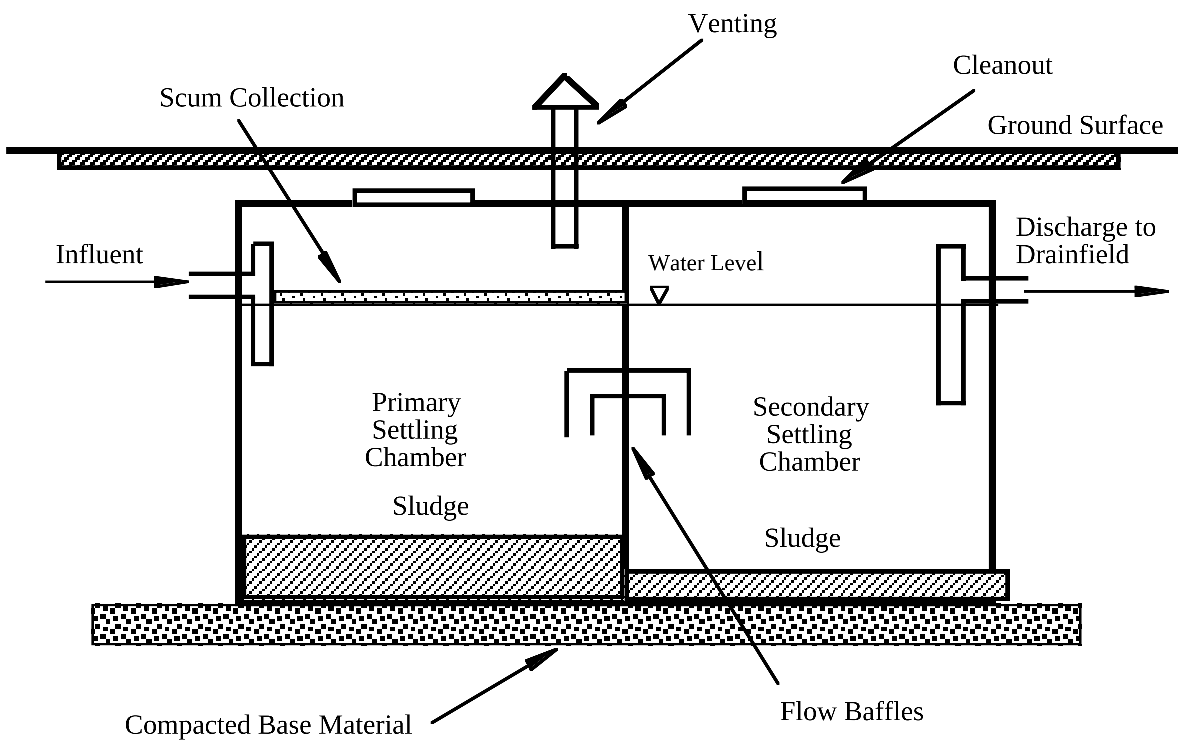

Figure 10.1. Typical Two-Chambered Septic Tank with Surface Venting.

Generally, to produce effluent quality that has a low BOD, TSS, and TN a more complex process approach is required. More complex systems involve the combination or sequencing of aerobic and anoxic processes to oxidize carbonaceous material and convert ammonia to nitrate and subsequently convert nitrate to nitrogen gas. Strictly anaerobic processes work slowly and are subject to frequent upsets because the microbes are sensitive to temperature changes and sensitive to many pollutants and chemicals found in household wastewaters. Treatment units with the potential to accomplish advanced treatment of nitrogen species must have some form of the elements of the system shown in Figure 10.2. This unit has simple sedimentation for primary treatment, an aerobic process (fixed film or suspended growth) for secondary treatment with nitrification, and an anoxic process for tertiary treatment. Constructed wetlands (Figure 10.3) are a fixed film system, which has the complexity in the biofilm to provide the required microbial processes to achieve advanced treatment of nitrogen.

The first step in evaluating any wastewater treatment technology is to understand the fundamental function of the various unit operations that make up the treatment train. Understanding the underlying science that drives the function of each operation, coupled to proper sampling and statistical analysis allows for the determination of the performance of each system. This approach is applicable to large systems treating millions of gallons per day of wastewater as well as small, onsite systems treating wastewater flows of 100 to 1,000 gallons.

10.1 Solids/Liquid Separation

The settling tank (pre-treatment tank or trash tank) in the alternative system performs the same function as a clarifier in the large municipal system. The settling tank unit operations of most interest for small systems are:

- Solid/Liquid Separation
- Trash Tank
- Primary Settling tank (trash tank or septic tank)
- Final Settling Tank

All readily settleable material is removed from the wastewater prior to the aeration basin. If this material is not removed, it becomes an oxygen demand and the biological degradation of this material will generate ammonia. The efficient degradation of both the BOD and the ammonia will require oxygen. If the oxygen transfer capability of the system is insufficient to provide for

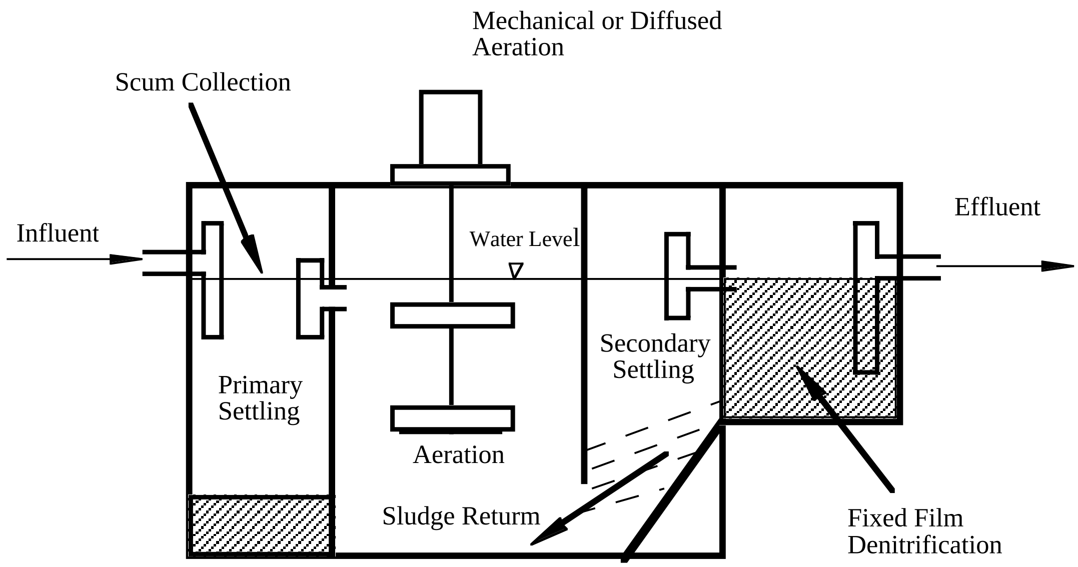

Figure 10.2. A Typical Aerated Alternative Onsite Wastewater Treatment System with Denitrification.

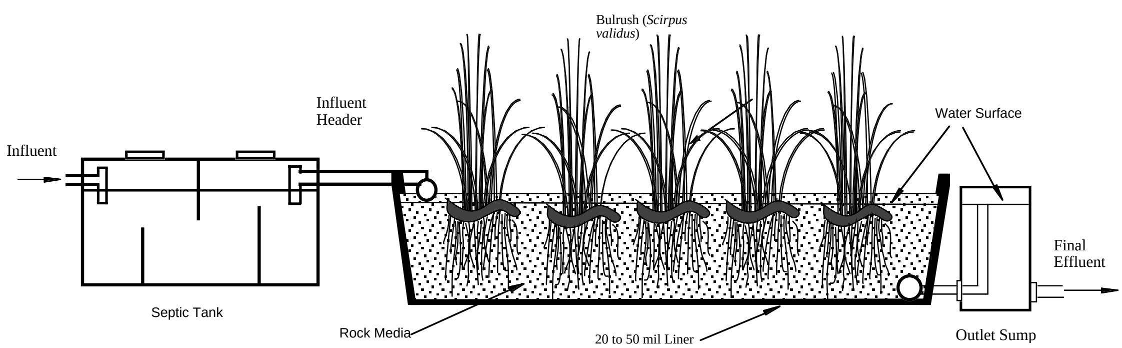

Figure 10.3. Typical Arrangement for an Onsite Submerged Surface Flow (SSF) Constructed Wetlands with Septic Tank Pretreatment.

the ammonia demand and the oxygen provided is insufficient to fully convert the ammonia to nitrate, the advanced treatment system will appear to create ammonia. A trash tank should be required for all alternative systems.

Since individual household waste is similar in many ways to conventional municipal wastewater, it is possible to use data from municipal systems to estimate removal efficiencies that might be expected from a trash tank based on primary clarifier design expectations. The removal percentage of both BOD and TSS are a function of the settling tank overflow rate. Typical overflow rates for primary clarifiers used in municipal wastewater treatment range from 600 to 1200 gpd/ft^2 (gallon per day per foot squared). The following table (10.1) gives typical removal efficiencies for municipal clarifiers as a function of overflow rate.

Table 10.1 Estimated Clarifier Removal Efficiency for Total Suspended Solids.

	Over Flow Rate, gpd/ft^2			
	1,200	1,000	800	600
	Over Flow Rate, m/d			
% Removal	48.9	40.7	32.6	24.4
TSS, %	54	58	64	68
BOD, %	30	32	34	36

It is possible to estimate the overflow rate for a trash tank using equation 10.1.

$$v_o = Q/A_s \qquad (10.1)$$

Where the overflow rate, v_o, is the terminal settling velocity of the smallest particle 100% removed in the settling tank, A_s is the surface area of the tank, and Q is the flowrate of waste water through the tank. It is anticipated that the average trash tank will have an overflow rate considerably smaller than the ones used in municipal treatment, and should therefore have a higher percent removal.

Consider a typical 3-bedroom home, and assume two people per room producing 100 gpd of wastewater resulting in total wastewater flow of 600 gpd. The Uniform Plumbing Code requires a 1,000 gallon tank, that is typically 3 feet deep. The surface area (A_s) will be about 45 ft^2 and with the flow rate (Q) of 600 gpd. Using the formula given above results in a calculate overflow rate: v_0 = 13.3 gpd/ft^2. Clearly, a trash tank in the system is important for taking a large portion of the waste load off from the biological treatment portion of the system.

10.2 Biological Systems

Biological wastewater treatment processes are used to transform solid, dissolved and colloidal pollutants into gases, cell material, and metabolic end products. These processes may occur in the presence or absence of oxygen. In the absence of oxygen (anaerobic and anoxic process), wastewater materials may be hydrolyzed and the resultant products fermented to produce a variety of alcohols, organic-acids, other reduced end products, synthesized cell mass, and gases including carbon dioxide, hydrogen, and methane. Further treatment of the effluents from anaerobic processes is normally required in order to achieve an acceptable quality for surface discharge. On the other hand, aerobic processes will generate high quality effluents containing a variety of oxidized end products, carbon dioxide, and metabolized biomass.

Every successful biological treatment system must have a minimum of two parts: conversion to biological solids and removal of those biological solids. There are two common measures of the degree or magnitude of organic contamination; these are biochemical oxygen demand (BOD) and chemical oxygen demand (COD). BOD is a measure of the soluble contaminant available to the microorganisms. It does not measure the material unavailable to the microorganism, such as recalcitrant organic structures, or toxic materials. COD is a measure of the total oxidizable carbon present. COD often includes organic material that is not available to the microorganisms. BOD can be partitioned as BOD_C and BOD_N. Both of these are measures of the amount of oxygen required to oxidize the soluble reduced contaminant present. The BOD_C is made up of reduced carbon. The BOD_N is predominantly NH_4 that will be converted to NO_3. The microbes that consume carbon grow fairly rapidly, but the microbes that consume the ammonia grow slowly. In addition, the microbes prefer the carbon material as a feedstock and will usually consume little of the ammonia until the BOD has dropped below 20 mg/L.

Nitrogen can occur as both organic nitrogen and urea nitrogen in raw wastewaters (Metcalf and Eddy, Inc., 1991). Organic nitrogen is contained in all organic materials found in wastewaters including feces and kitchen wastes and undergoes the microbially driven process of ammonification to release ammonia to the wastewater. Urea, the major component of urine, hydrolyzes rapidly to form ammonia and carbon dioxide in the presence of microorganisms containing the enzyme urease as shown in equation 10.2. The breakdown of urea to form ammonia may take just a few minutes while the release of ammonia from organic nitrogen sources may take several days or months. Thus, the nitrogen in most raw wastes is a combination of these nitrogen forms with ammonia the dominant species.

$$H_2NCOH_2N + H_2O \xrightarrow{\textit{Urease}} CO_2 + 2NH_3 \qquad (10.2)$$

Once ammonia is present it can be removed biologically from wastewater via the process of nitrification/denitrification. Nitrification is a strictly aerobic, two-step microbial process, which converts ammonia to nitrite and then to nitrate as shown in equations 10.3 and 10.4. *Nitrosomonas* oxidizes ammonia to the intermediate product nitrite, and *Nitrobacter* converts

$$NH_4^+ + 1.5\ O_2 + 2\ HCO_3^- \xrightarrow{\textit{Nitrosomonas}} NO_2^- + 2\ H_2CO_3 + H_2O \qquad (10.3)$$

$$NO_2^- + 0.5\ O_2 \xrightarrow{\textit{Nitrobacter}} NO_3^- \qquad (10.4)$$

nitrite to nitrate. Stoichiometrically it takes 4.3 mg/L of O_2 and 8.64 mg/L (as $CaCO_3$) of alkalinity to convert 1 mg/L of ammonia to nitrate (as N) (Kadlec and Knight, 1996). If oxygen, alkalinity or both are limiting, the reaction will not proceed. In addition, nitrification can be limited by temperature, salinity, presence of a carbon source, and pH (USEPA, 1993b). Nitrifiers grow slowly and rates of ammonia conversion are lower than the rate of organic material conversion.

The degradation of organic material competes with the nitrifying bacteria for oxygen and rates of nitrification will be inhibited by the presence of quantities of BOD_5 above 20 mg/L. Consortia of microorganism reduce organic matter according to Equation 10.5. Organic material can occur as both dissolved and degradable particulate material, both of which can exert a

$$COHNS + O_2 + \text{nutrients} \xrightarrow{} CO_2 + NH_3 + C_5H_7NO_2 + \text{end products} \quad (10.5)$$

(bacteria) (new bacteria matter)

demand for oxygen in a system. The degradation of TSS is generally assumed to exert about 0.5 mg of BOD_5, for each mg of TSS degraded. It is critical to the nitrification process that BOD_5 be reduced to concentrations that will not inhibit the growth of nitrifiers and that sufficient oxygen is present to allow the nitrification process to proceed to the end products.

Denitrification results in the removal of nitrate by conversion of nitrogen to nitrogen gas accomplished under anoxic conditions by a diverse group of facultative bacteria (Metcalf and Eddy, Inc., 1991). Dissimilitory nitrate reduction is a two-step process with the first step involving the conversion of nitrate to nitrite as shown in equation 10.6. The second step is the production of nitric oxide, nitrous oxide, and then nitrogen gas as in equation 10.7. The availability of a carbon source is critical for completion of this process. The carbon source

$$NO_3^- + 0.33\ CH_3OH \longrightarrow NO_2^- + 0.67\ H_2O \quad (10.6)$$

$$NO_2^- + 0.5\ CH_3OH \longrightarrow 0.5\ N_2 + 0.5\ CO_2 + 0.67\ H_2O + OH^- \quad (10.7)$$

is oxidized and donates electrons while nitrate gains electrons and is reduced to nitrogen gas and released to the atmosphere. Many different compounds can serve as a carbon source, but not all compounds result in an efficient conversion of nitrate to nitrogen gas. This process will not occur unless there is enough food present for the microorganisms. If the BOD was low enough to induce nitrification, there will not be enough food available to drive the denitrification. This means that a supplemental food source must be added. This can be wastewater that has by-passed the biological treatment process or it can be an added carbon source such as methanol or acetate.

There are two types of biological reactors that are currently being marketed for advanced treatment fixed film systems and suspended growth systems. The majority of the commercial systems are suspended growth systems in which the microbial biomass is suspended in the water column by the mixing the aerator provides. In the fixed film systems the microbial biomass is attached to some inert media. The fixed film system has the advantage of keeping the microbes in the reactor. This is a notable advantage if you are attempting to convert ammonia to nitrate, since these microbes grow slowly. It is also an advantage in encouraging denitrification since there are anoxic zones within the biofilm.

10.3 Suspended Growth Systems

Almost every commercial suspended growth system uses extended aeration. This process is a modification of the activated sludge process where a high concentration of microorganisms (MLVSS) are maintained in an aeration tank, followed by decanting and separation and recycle of all or a portion of the biomass back to the aeration tank. There are a variety of proprietary extended aeration package plants available on the market today for onsite application. The process may be operated in a batch or continuous flow mode, and oxygen is supplied by either diffused or mechanical aeration. Positive biomass return to the aeration tank is normally employed, but wasting of excess solids varies widely between manufactured units.

Extended aeration processes are more complex than septic tanks, and require regular operation and maintenance. The aeration system requires power, and some noise and odor may be associated with it. In extended aeration package plants, long hydraulic and solids retention times (SRT) are maintained to ensure a high degree of treatment at minimum operational control, to provide some protection against hydraulic or organic overloading, to allow for nitrification, and to reduce net sludge production. At one time extended aeration was thought to provide a biological system that would require no sludge wasting because endogenous respiration would digest sludge as quickly as it was produced. Operation practices have proven that these systems require regularly scheduled wasting of solids. Not wasting accumulated solids can result in SRT increases to a point where the clarifier can no longer handle the solids producing high TSS effluent concentrations that can plug drainfields. Treatment performance normally improves with increasing hydraulic retention time, but excessive solids build-up will result in high suspended solids washout. Optimizing these two requirements is one of the biggest operational problems with extended aeration units.

Dissolved oxygen concentrations in the aeration tank should be maintained at 2 mg/L or greater to insure good treatment performance and a good settling sludge. Normally, onsite extended aeration plants supply an excess of dissolved oxygen due to minimum size restrictions on blower motors or mechanical drives. An important element of most aeration systems is the mixing provided by the aeration process. Most package units provide sufficient mixing to ensure good suspension of solids and mass transfer of nutrients and oxygen to the microbes. Failure of the aeration system is the biggest problem associated with aerobic system (WPCF 1977).

Wastewater temperature and characteristics may also influence performance of the process. Excess amounts of certain cleaning agents, greases, floating matter, and other detritus can cause process upsets and equipment malfunctions. Process efficiency increases with increasing temperature and can decrease dramatically with decreasing temperatures.

The clarifier is an important part of the process because the biomass must be properly separated from the effluent or the SRT of the sludge, cannot be maintained and excessive solids will be discharged to the effluent. Clarifier performance depends upon the settleability of the biomass, the hydraulic overflow rate, and the solids loading rate. Hydraulic surges can result in serious clarifier malfunctions. As mentioned previously, high solids loadings caused by accumulation of MLVSS can result in eventual solids carryover. Excessively long retention times for settled sludges in the clarifier may result in gasification and flotation of these sludges. Scum and floatable material not properly removed from the clarifier surface will greatly impair effluent quality as well.

Most extended aeration package plants designed for individual home application range in capacity from 600 to 1,500 gal (2,270 to 5,680 L). This includes the aeration compartment, settling chamber, a pretreatment compartment, and some form of denitrification. Based upon average flows from households, this volume will provide total hydraulic retention times of several days. Some aerobic units provide a pretreatment step to remove grease, trash, garbage grindings, and other solids. Pretreatment devices can include trash traps, septic tanks, comminutors, and aerated surge chambers. The use of a trash trap or septic tank preceding the extended aeration process reduces problems with floating debris in the final clarifier, clogging of flow lines and aerators, and plugging of pumps.

Aerobic package plants are typically designed as continuous flow systems. The simplest continuous flow units provide no flow equalization and depend upon aeration tank volume and/or baffles to reduce the impact of hydraulic surges. Some units employ more sophisticated flow dampening devices, including airlift or float-controlled mechanical pumps to transfer the wastewater from aeration tank to clarifier. Still other units provide multiple-chambered tanks to attenuate flow.

Oxygen is transferred to the MLVSS by means of diffused air, sparged turbine, or surface entrainment devices. When diffused air systems are employed, low head-blowers or compressors are used to force the air through the diffusers placed on the bottom of the tank. The sparged turbine employs both a diffused air source and external mixing, usually by means of a submerged flat-bladed turbine. The sparged turbine is more complex than the simple diffused air system. There are a variety of mechanical aeration devices employed in package plants to aerate and mix the wastewater. Air is entrained and circulated within the mixed liquor through violent agitation from mixing or pumping action. Oxygen transfer efficiencies for these small package plants are normally low (0.2 to 1.0 lb 0_2 /hp-hr) (3.4 to 16.9 kg 0_2/MJ) as compared with large-scale systems due primarily to the high power inputs to the smaller units (USEPA 1980). Normally, there is sufficient oxygen transferred to produce high oxygen levels. In an attempt to reduce power requirements or to enhance nitrogen removal, some units employ cycled aeration periods. Care must be taken to avoid the development of poor settling biomass when cycled aeration is used.

Mixing of the aeration tank contents is also an important consideration in the design of oxygen transfer devices. Rule of thumb requirements for mixing in aeration tanks range from 0.5 to 1 hp/1,000 ft^3 (13 to 26 kw/1,000 m^3) depending upon reactor geometry. Commercially available package units reported to deliver mixing inputs ranging from 0.2 to 3 hp/1,000 ft^3 (5 to 79 kw/1,000 m^3) (USEPA 1980). Deposition problems may develop in those units with the lower mixing intensities.

The clarifier is critical to the successful performance of the extended aeration process. A majority of the commercially available package plants provide simple gravity separation. Weir and baffle designs have not been given much attention in package units. Weir lengths of at least 12 in. (30 cm) are preferred (10,000 gpd/ft at 7 gpm) (127 m^3/d/m at 0.4 L/sec) and sludge deflection baffles should be included as a part of the outlet design (USEPA 1980). The use of gas deflection barriers is a simple way to keep floating solids away from the weir area.

Upflow clarifier devices have also been employed to improve separation. Hydraulic surges must be avoided in these systems to avoid solids washout. Filtration devices have also been employed in some units, but while filters may produce high-quality effluent, they are very susceptible to both internal and external clogging. In addition they may require a higher degree of maintenance. The behavior of clarifiers is dependent upon biomass settling properties, solids loading rate, and hydraulic overflow rates. Design peak hydraulic overflow rates should be less than 800 gpd/ft^2 (32 $m^3/d/m^2$); and at average flow design values normally range from 200 to 400 gpd/ft^2 (8 to 16 $m^3/d/m^2$). Solids loading rates are usually less than 30 lb/ft^2/d (145 kg/m^2/d) based upon average flow and less than 50 lb/ft^2/d (242 kg/m^2/d) based upon peak flows, (USEPA 1980).

Once separated from the treated wastewater, the biomass must be returned to the aeration tank or be wasted. Air-lift pumps, draft tubes working off the aerator, and gravity return methods are normally used. Rapid removal of solids from the clarifier is desirable to avoid gas generation and possible flotation of solids and loss of biomass. Positive sludge return should be employed in package plants since the use of gravity return systems has generally proved ineffective.

Most onsite package plants do not provide for routine wasting of solids from the unit. Some systems however, do employ an additional chamber for aerobic digestion of wasted sludge. Wasting is normally a manual operation whereby the operator checks mixed liquor solids and wasted sludge when mixed liquor concentrations exceed a selected value. In general, wasting should be provided once every 8 to 12 months (USEPA 1980). Removal of floating solids from clarifiers has normally been ignored in most onsite package plant designs. Since this material results in serious deterioration of the effluent, efforts should be made to provide for positive removal of this residue. Reliance on the owner to remove floating scum is unrealistic.

Generally, extended aeration plants produce a high degree of nitrification since hydraulic and solids retention times are greater than 10 days. Reductions of phosphorus are normally less than 25%. The removal of indicator bacteria in onsite extended aeration processes is highly variable and not well documented. Reported values of fecal coliforms appear to be about 2 orders of magnitude lower in extended aeration effluents than in septic tank effluents.

10.4 Fixed Film Systems

Fixed film systems employ an inert media of various types that provides attachment sites for microorganisms. The wastewater comes in contact with this fixed film of microorganisms either by pumping the water past the media or by moving the media past the wastewater to be treated. Oxygen may be supplied by natural ventilation or by mechanical or diffused aeration within the wastewater. Typical fixed film reactors are trickling filter (gravity flow of wastewater downward), the upflow filter (wastewater pumped upward through the media), and the rotating biological contactors (RBC).

The trickling filter has been used to treat wastewater for many years. Modern filters today consist of towers of media constructed from a variety of plastics, stone, or redwood laths into a number of shapes (honeycomb blocks, rings, cylinders, etc,). Wastewater is distributed over the surface of the media and collected at the bottom through an underdrain system. Oxygen is normally transferred by natural drafting, although some units employ blowers. Treated effluent is settled prior to being discharged or partially recycled back through the filter. In an upflow filter, wastewater flows through the media and is subsequently collected at an overflow weir. Oxygen may be transferred to the biomass by means of diffusers located at the bottom of the tower or by surface entrainment devices at the top.

The Rotating Biological Contactor (RBC) employs a series of rotating discs mounted on a horizontal shaft. The partially submerged discs rotate at rates of 1 to 2 rpm through the wastewater. Oxygen is transferred to the biomass as the disc rotates from the air to the water phase. Recirculation of effluent is not normally practiced.

Generally, onsite fixed film systems are less complex than extended aeration systems and should require less attention; if designed properly they should produce an effluent of equivalent quality. There are no significant physical site constraints that should limit their application, although local codes may require certain setback distances. The process is more temperature sensitive than extended aeration and should be insulated as required. Rotating biological contactors should also be protected from sunlight to avoid excessive growth of algae that may overgrow the plate surfaces.

Onsite fixed film systems include a variety of proprietary devices. Settling and/or screening to remove materials that might interfere with the operation should precede all fixed film systems. Hydraulic loadings are normally constrained by biological reaction rates and mass transfer. Organic loading is primarily limited by oxygen transfer within the biological film. Excessive organic loads may cause anaerobic conditions resulting in odor and poor performance. Dissolved oxygen in the liquid should be at least 2 mg/L. Recirculation is not normally practiced in package fixed film systems since it adds to the degree of complexity and is energy and maintenance intensive. However, recirculation may be desirable in certain applications where minimum wetting rates are required for optimal performance. The production of biomass on fixed film systems is similar to that for extended aeration. However, biomass may slough off in large amounts over time. Proper clarification is required to avoid solids washout. Very often, accumulated sludge is directed back to the septic tank for storage and partial digestion. However sludge wasting in these systems is still necessary and may be required more frequently depending on the design. Generally heterotrophic bacteria form on the upstream end of the film material and nitrifiers grow on the downstream film surfaces.

Recently, many systems have incorporated fixed film technology for denitrifying effluent from a variety of aerobic systems. Fixed films depend on a high rate of flow past the film to maintain aerobic conditions. Lower flow conditions results in an anoxic or anaerobic film that is an excellent environment for denitrifying bacteria. Many systems have used packed plastic rings, biofilter balls, or cross flute corrugated block (Biodek) as the growth media. Wastewater flow can be upflow, downflow or horizontal flow.

Chapter 11 - Whitewater System

11.1 Site Description

The system tested was a Whitewater Aerobic Treatment Unit; Model DF50-FF manufactured by Delta Environmental Products, Inc. This system is characterized as a suspended growth biological (SGB) treatment system designed to treat 500 gallons per day of domestic wastewater. A local manufacturer representative provided the unit. An approved local onsite system installer installed the unit. The test site was a three-bedroom house located in the South Valley area of Bernalillo County/Albuquerque off of Isleta road. The house was estimated to have three full time occupants, has a water softener for a portion of the water flow, and a garbage grinder is installed. The water softener backwash at this residence does not flow to the onsite system, but is diverted to an outside French drain. The previous system was a septic tank and conventional leachfield. Sometime in early 1997 this system began to fail as indicated by odors and ponding in the backyard.

When viewed by the research team the septic tank had collapsed and the interior was visible from ground surface. An installer was contacted and bids were received for the installation of an alternative system. Additionally because the installation was in an area with a high water table (about 4 feet to groundwater) a mound leachfield system was required for effluent disposal. This was included in the bid price for the treatment unit installation. The cost of installation, including disposal of excavated septic tanks and partial removal of old leachfield, was $13,000. Permits were applied for and obtained by the installer and the system installation was completed in September 1997. The system layout with sample sumps and required setbacks is shown in Figure 11.1. Sampling was set up using methods outlined in the previous methods section.

11.2 System Description

The system tested was a Whitewater Aerobic Treatment Unit; Model DF50-FF manufactured by Delta Environmental Products, Inc as shown in Figure 11.2. This system is characterized as suspended growth biological (SGB) treatment system designed to treat 500 gallons per day of domestic wastewater. The unit has a cylindrical shape with a conical internal

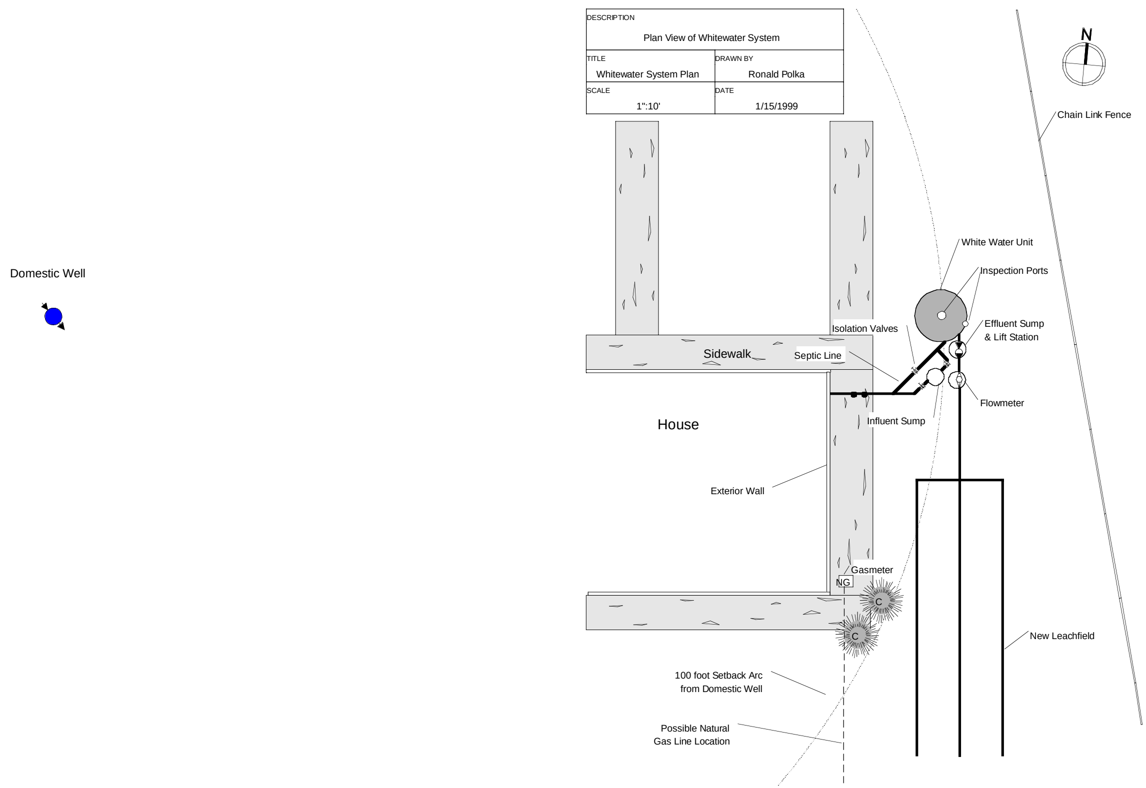

Figure 11.1. Whitewater System Plan View.

Figure 11.1. Whitewater System Plan View.

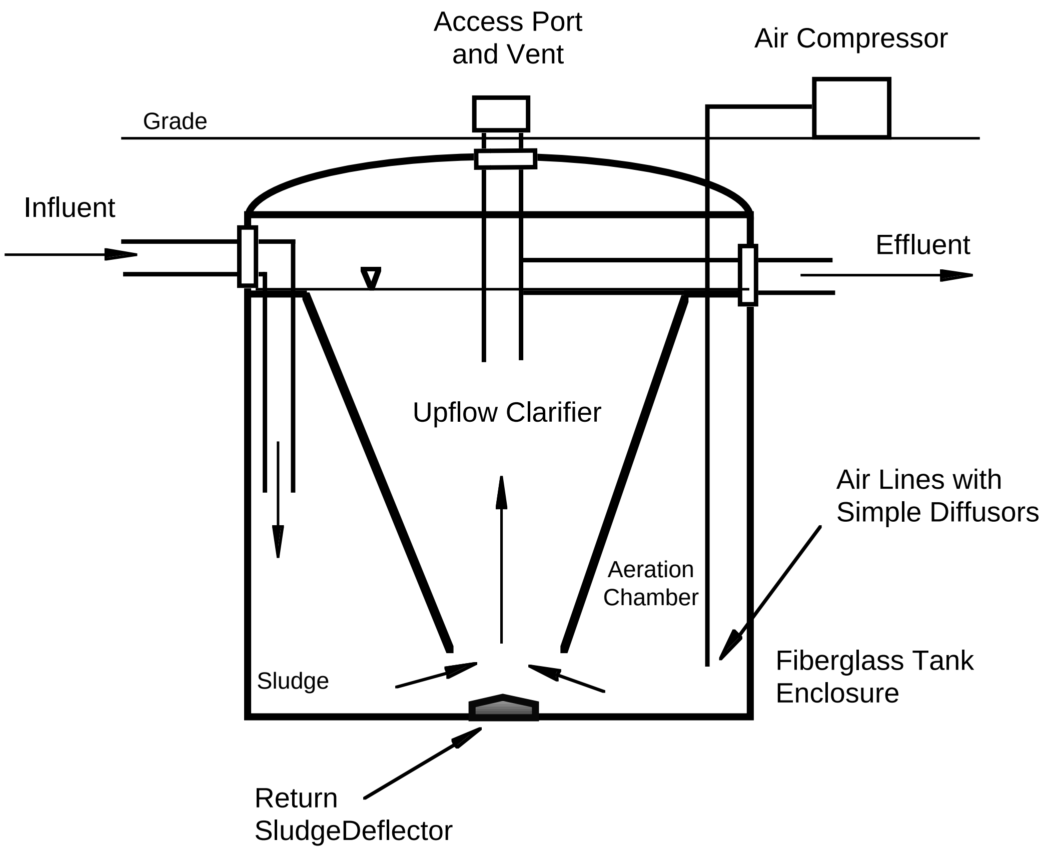

Figure 11.2. Side View of the Whitewater Onsite Wastewater Treatment System.

clarifier with gravity sludge return to the aerobic system. The system is constructed of fiberglass. This unit did not come with a pretreatment trash tank. The unit is the standard unit provided by the supplier to the Albuquerque area.

11.3 Process Description

The Whitewater Aerobic Treatment Unit manufactured by Delta Environmental Products, Inc. is suspended growth biological (SGB) treatment system. This company manufactures nine models with various material configurations for flowrates of 400 to 1,500 gpd. The models have all been tested and approved under NSF Standard 40 -Residential Wastewater Treatment Systems. The company has representatives listed in twenty-seven states including New Mexico. The treatment system consists of influent piping, a fiberglass treatment tank with two internal compartments, and effluent piping to a disposal drainfield. An optional pretreatment tank or trash tank for solids collection can be provided (DEP, Inc.). The system tested in this study did not have this optional tank.

The influent piping flows by gravity from the residence to the unit, but under some conditions a pump station may be required to transfer raw wastewater to the treatment unit. This arrangement would require a grinder pump. The circular treatment unit receives the raw wastewater into an outer chamber. This chamber is aerated by five drop tube aerators connected to a small blower. These aerators are simple open-ended tubes that transfer air to the mixed liquor suspended solids (MLSS) for oxygen and mixing. This configuration exposes raw solids and sewage to the activated sludge or aerated solids in a single step. This mixing is constant and produces an aerated effluent. The MLSS must allow for BOD and ammonia oxidation. High SRT's are obtained by constant recirculation of the generated solids from the integral clarifier. The clarifier is an upflow configuration that provides contact between the settling solids and the liquid as it flows from the treatment unit. The company does not state specifically that the unit will remove nitrogen either via nitrification or denitrification. But they provide data that indicates both species should be removed by the process. If this is the case then nitrification, denitrification, and BOD removal is taking place simultaneously within regions of the MLSS. It is typical to expect BOD and ammonia removal in an extended air process, but achieving denitrification within the same processes is not a typical design approach.

The Whitewater Aerobic Treatment Unit; Model DF50-FF manufactured by Delta Environmental Products, Inc. has a total system volume as installed of 909 gallons and aeration volume of 720 gallons. At a design flow of 500 gpd this results in a detention of times of 1.81 days and 1.44 days for the total system and the aeration basin respectively. The final clarifier has a volume of 189 gallons and at design flow has a detention time of 0.38 days. The manufacturer suggests that an MLSS of 3,000 to 5,000 mg/L should be maintained in the system.

11.4 Unit Installation

The manufacturer’s installation literature (Delta Environmental Products, Inc., 1995) was used to document this section of the report. The research team did observe the installation of this unit.

11.4.1 Manufacturer's Recommendations

The manufacturer's instructions suggest that all system installations be carried out by a certified licensed installer. Prepare an excavation, having a diameter approximately one foot larger than the tank and a depth that will allow approximately 3 inches of the inspection port to extend above normal ground level. Backfill with a 6 inch layer of sand or gravel if otherwise unable to provide a smooth, level, compact base. The manufacturer recommends that the hole be roped off in some fashion to prevent injury to passersby. Utilizing lifting lugs provided, place the plant in the excavation so that the inlet and the outlet line up with the sewer piping. The inlet line should slope down toward the plant and the outlet line should slope down away from the plant. The Plant should be level within 1/2 inch, edge to edge. Position inlet and outlet lines and make connection as necessary, depending upon the construction materials. The inlet line should be inserted and glued into the inlet elbow and the discharge line should be inserted and glued into the outlet coupling. Note: Open inspection port and make sure discharge tee assembly is level and centered in clarifier prior to attaching discharge piping. Fill the tank with water until water flows from the discharge before backfilling. Backfill around plant, up to the bottom of the discharge connections, taking care not to damage the surface coating or dislodge the piping. If the surface is damaged it can be repaired with bitumastic coating available from Delta.

Install the air blowers in a clean, well-ventilated area, such as a tool room, garage, or an exterior blower housing, within 100 ft. of the unit. The blower should be installed near the control panel. Mount the control panels in an area where the alarm can be heard and be readily observed. A 3 wire grounded outlet is required for safety and all electrical work shall be done according to NEC requirements (Note following instructions and precautions). The control panel is rated for indoor and outdoor use and contains a fused receptacle into which the compressor is plugged. An electrical malfunction in the compressor or wiring to the compressor will cause the fuse to blow. The control panel also contains a pressure switch, audible alarm, and battery. Loss of air pressure caused by loss of power or compressor system malfunction will cause the alarm to sound.

Remove the front cover of the control panel and attach control panel to a suitable mounting surface using all four mounting holes on back of box. Use proper screws of sufficient length to insure a secure and permanent mounting. Control box is rated for outdoor service, however, do not place it where it can be immersed in rising water or where run-off water such as from a roof will fall on it. Also, do not mount it where it is subject to wetting from sprinklers, hoses, etc. The control box must never be connected to a circuit that is not properly grounded. Never plug the unit into a non-grounded receptacle or a receptacle that has a 2 pole to 3 pole grounding adapter attached. If there is doubt, have a qualified electrician check for proper grounding. The control box must be connected to an electric source equipped with a ground fault interrupter (GFI) circuit breaker or receptacle. A standard receptacle can be replaced with a GFI receptacle that can be obtained from an electrical supply dealer. After the box is properly mounted, and with power source disconnected, insert the compressor plug through the hole in the bottom on the control box and plug it into the fused receptacle inside the box. Pull the pressure tubing out of the same hole. Insert the compressor cord and pressure tubing through the hole split sealing plug. Insert the plug into the control box hole making sure the metal is fitted properly into the plug groove. The plug protects the cord and tubing from sharp edges and also seals the box from insect intrusion. Make sure there is ample slack in the compressor cord and tubing, inside and outside of the control box such that the cord and tubing exit the box vertically and does not impose strain on the plug or tubing connection. Strain may cause the sealing plug to dislodge over time.

The control box is equipped with an electric cord to allow for easy installation, however, local codes may require the unit to be hardwired directly to the source. In this case the electric cord can be removed and the entrance hole used for approved conduit or wire attachments. If the receptacle cover is removed to rewire, it is very important that both factory screws used to mount the receptacle cover to the inside handy box are replaced and tightened. Also, the inside handy box is secured to the back of the control box with special screws and outside serrated washers to insure a good ground connection. It is important that these screws also be tight at all times. Connect the pressure tube to the 1/8 barb-fitting in the air piping system. Install 1 inch schedule 40 PVC piping between blower and treatment unit. A minimum of 12 inches of ground cover is recommended.

Plug the control panel cord into service outlet to start the compressor. Check the air piping joints for leakage using a soapy water solution. Repair, if necessary and then carefully backfill airline and inlet and discharge piping and cover plant to level grade. Plant is ready to receive incoming sewage. After the home treatment unit is full of water, install a fresh 9-volt battery in the alarm circuit. Test alarm circuits by momentarily disconnecting the power from the compressor and allowing air pressure to decrease. Alarm should sound. Plug compressor back in and alarm should stop sounding. Replace cover on control box. Change battery every 6 months or after an alarm situation has occurred. In the event that a fuse blows, replace with a 2 amp Type T, Dual element time delay. The distribution of air to all drop lines must be uniform. If airflow is not evenly distributed, an adjustment can be made by raising or lowering the individual air drop line. Whenever possible, spend time with your customer and review operating instructions and be sure that the customer has a manual to keep. This saves valuable time avoiding return visits. Retain these instructions for future reference.

11.4.2 Observed Conditions

Our research team did observe this unit and was involved in the installation and training provided for both the installer and the homeowner. The manufacturer was present for about 60 percent of the installation. The manufacturer provided the installer with an installation booklet and provided some onsite advice particularly about leveling tolerances for the unit. The manufacturer provided information about the installation of the blower unit and explained its operation to the installer. The manufacturer also provided a verbal explanation of the system to the homeowner and provided the homeowner with a maintenance and operation booklet. All of this assistance was provided both orally and with written material and was presented very well. But these short discussions were the extent of the training provided and no ongoing or follow-up discussions were conducted.

No significant problems were observed with the installation and the installation did appear to follow guidelines set out in the installation manual. The unit was installed in a shallow water table setting with water being visible at about a three to four foot depth. The installer excavated the area and placed the unit into the hole. No attempt was made to pump the excavated pit of freestanding water that was about a foot deep. The unit was placed into the pit with water and leveled with a simple bubble level. The unit was partially filled with water and then backfilled around the unit with the level checked at each step. Then final backfill was completed. A better option might have been to use a de-watering pump to remove the stand water and then add some gravel substrate to act as a footing for the unit and then place the unit. Certainly cost is a factor but little or no detail was provided from the manufacturer's representative.

11.5 System Operation and Maintenance

The manufacturer’s recommendations for maintenance to be performed by the homeowner are provided in the following sections.

11.5.1 Manufacturer’s Recommendations

Once the unit has been installed, (see installation instructions) the unit will operate with a minimum amount of attention. Please reference the system’s Data Plates that are located on aerobic tank 24" cover, air pump, and the alarm panel in the event that a problem arises or service is required. The following should be accomplished as checks for system failure:

- Observe the warning device which comes on when the power to the air pump has been interrupted or when the air supply system has malfunctioned. If the alarm is activated, replace the battery with a new one and check for a blown fuse or thrown circuit breaker. Check air pump to be sure it is operating. Once accustomed to the soft humming sound of a properly operating unit, any unusual noise is an indication of malfunction. If an unusual noise is detected or total failure is observed, call your local dealer for service.

- The battery in the alarm panel should be checked to ensure that its power level is adequate. If the pump is disconnected and the alarm does not sound after a period of 5 to 10 minutes, the dead battery should be replaced with a new one. If the alarm still does not sound after replacing the battery, refer to the troubleshooting guide.

- The battery should be replaced every 6 months.

- Weekly: Check the treatment plant for offensive odor. If such a condition should develop, call service.

- Every 3 Months: The air filter on the air pump should be cleaned. Rinse with warm water if necessary. (See installation instructions). Do not use oil or other solvents. To keep maintenance to a minimum and ensure high effluent quality, the following items should not be permitted to enter the system.

- Strong disinfectants or bleaches, other than small amounts normally utilized in day to day cleaning and laundry (be conservative). Laundry detergents recommended for use are low-sudsing, low phosphates and biodegradable, such as Gain, Arm and Hammer, All, Fresh Start, Dash Bright.

- Do not discharge any of the flowing items into the sewer:
    - Discharge from water softener
    - Any type of oils, greases, or other chemical wastes
    - Disposable baby diapers and wipes

- Sanitary napkins, condoms or other similar items
- Hair, bandages, rags, or string
- Latex, plastic, or metallic objects
- Coffee grounds or cigarette butts
- Mud or sticks
- Paper towels, napkins, or Kleenex
- Tidy Bowl type products
- Beer waste or any other rich liquids

- Garbage disposal should be used sparingly, not as a method of disposing all solid food waste. In order to ensure good plant operation, food waste should be disposed of in the garbage container, or a compost pit.

The System is designed to handle domestic wastewater and nothing else should go into it. For treatment of anything other than domestic wastewater contact Delta Environmental Products, Inc.

System Warnings: The proper operation of this or any other home sewage system depends upon proper organic loading and the life of the microorganisms inside the system. Delta is not responsible for the in-field operation of a system, other than the mechanical and structural workings of the plant itself. We cannot control the amount of harsh chemicals or other harmful substances that may be discharged into the system by the occupants of a household, we can only provide a comprehensive owner's manual that outlines substances that should be kept out of the system. Hydraulic overloading (flows in excess of design flow), may cause the sewage treatment system not to perform to the fullest capabilities.

Ants have been shown to be destructive to the air pump. Regular care should be taken to prevent infestation of ants near the system. Damage or destruction by ants is not covered under manufacturer’s warranty.

Your State or Local Health department may require other pieces of equipment to function separately or in conjunction with equipment manufactured by Delta Environmental Products. Delta Environmental Products is not responsible for the mechanical or electrical safety of equipment it does not manufacture or supply with its aerobic treatment unit. Particular care should be used in evaluating the electrical or mechanical safety of equipment manufactured by others. This may include but not be limited to electrical control panels or air pumps.

If an electrical GFI receptacle has not been installed for checking air distribution system during installation, use an extension cord to run the air pump. Never leave the extension cord plugged in. Remove it after testing is completed.

Due to a possible fire hazard, DO NOT plug into service equipment on power pole and DO NOT use extension cords. All electrical work performed by the installer or others must be in accordance with the National Electrical Code and Local Codes.

Solids Removal: The Whitewater Treatment System is designed to provide years of trouble free operation. Determination of the need for solids removal can be done through a simple test. A one quart sample should be pulled from the aeration tank and can be done so through the 4" sample port. Allow the sample to settle in a clear one-quart jar for one hour. If the solids content exceeds 60% of the total volume after settling, the treatment plant should be pumped out. Call your local authorized sewage disposal service to have the tank contents pumped out and disposed of properly.

The method of pumping out should be as follows. Remove any floating solids by skimming. The air pump must be operating to keep the solids in suspension. Pump out two thirds of the tank volume with the suction pipe opening being placed at the tank bottom. After the pump-out process is complete, fill the tank with fresh water to normal operating level. Refer to the Installation Instructions to get the treatment plant back into operation. Should indication of improper operation be observed at any point in time, contact your local distributor.

Seasonal Use Guidelines: These guidelines are for conditions as outlined below and apply for systems that are used periodically as indicated. Site conditions not covered by the following must be forwarded to Delta for recommended guidelines to meet the particular site conditions.

System not in use for more than one month and less than three months - Electrical power is left on and there are no frost conditions. Leave air pump on and system running.

System not in use more than three months - Electrical power is turned off and there are not frost conditions. While system is operating with the air pump on, remove all material and liquid from tank. Refill with clean water. Turn off air pump.

System not in use more than three months - Electrical power is on and there are not frost conditions. Leave air pump on and system running or, while system is operating with the air pump on, remove all material and liquid from tank. Refill with clean water and turn off air pump.

System not in use - Electrical power is turned off and there are frost conditions. While system is operating with the air pump on, remove all material and liquid from tank. Turn off air pump. If high ground water is present, fill with clean water. If no ground water is present, leave tank empty.

11.5.2 Observed Conditions

Operation and maintenance for this system proved to be straight-forward with the exception of a recurring problem with supplying air to the unit. During the initial phase of sampling it was noted that the dissolved oxygen (DO) levels of effluent samples were quite low, providing an average of 0.8 mg/L during the first six months of operation. It is believed that aeration of the system was not working at this time. During a sampling event on April 2, 1998 the DO level was observed to be 3.3 mg/L, up substantially from the previous eight sample visits. It was noted that aeration of the system was in operation. On the next sample visit on April 16 it was noted that the effluent DO was once again low, back down to 0.4 mg/L. An examination of the system indicated a problem that was initially caused by the homeowner but is ultimately due to poor system design. The system controller and air compressor were housed in a plastic doghouse for weather protection. The homeowner was in the process of spring cleaning in the yard, pulling weeds and preparing the soil for the seeding of grass. During the course of this work the doghouse was inadvertently moved slightly. This movement disturbed the relative position of the compressor outlet and air line leading to the unit. The air line was a buried 3/4" PVC pipe. The compressor outlet was a straight pipe that was connected to the air line with a short piece (about 4") of rubber hose held in place with two self-tightening spring hose clamps. The jarring of this connection while under pressure caused one end to pop off, thereby stopping air flow to the system. It was fixed by the sampler and the spring hose clamps were replaced with worm drive hose clamps that can be tightened securely. An air line under pressure should not be joined by a smooth pipe termination and spring clamps. This seems to be a poor design by the compressor manufacturer. A barbed fitting and worm drive hose clamps would be a much better choice at a negligible cost increase.

After the air line was fixed the system operated at good effluent DO levels, averaging 3.3 mg/L until the sample visit on July 9. At this time it was again noted that the effluent DO level was low, 0.3 mg/L. An examination of the system revealed that the alarm buzzer was activated. The sampler could not determine the cause of the problem so on the following day a call was placed to the system manufacturer's representative to report the problem. The representative made a site visit and corrected the problem by reconnecting a small pipe to the alarm pressure indicator. After that the system effluent DO levels remained at relatively high values, averaging 2.6 mg/L for the next six months. A design weakness is noted with the system alarm in that it is barely audible unless one is quite close to the unit and the surroundings are quiet. It is driven by a 9-volt battery and does not have an output that attracts attention. With this controller and alarm installed inside the doghouse it could not be heard until the doghouse was removed.

In two instances within a year of installation this unit experienced aeration system failures that were not noted by the homeowner. If there had not been a sampling program in progress the air deficiency may not have been discovered for a long time. It is recommended that the manufacturer review the air system design to correct these shortcomings. A more effective means of alerting the homeowner to aeration problems needs to be devised. It can be observed that a considerable degradation in system performance occurred during these periods of depressed DO levels. On the other hand, it must also be noted that the manufacturer's representative, when contacted about problems with the system, responded in a timely manner to correct them.

11.6 Reported System Performance

This extended aeration suspended growth system was designed according to the manufacturer to remove BOD, TSS, and the various forms of nitrogen from typical domestic wastewater. The manufacturer provided data regarding the performance of the system and Model DF40-M has been tested by the NSF International Inc. The results of the NSF test data using NSF test Standard 40 shown in Table 11.1 indicated an average influent and effluent BOD_5 of

Table 11.1 NSF Performance Data for Whitewater Model DF40-M from NSF Test Standard 40 (NSF 1993).

	Influent		Effluent		
Parameter	**Mean**	**Range**	**Mean**	**Range**	**Percent Removal**
BOD_5	173	80 - 360	6	5 - 20	96.5
TSS	189	100 - 440	7	5 - 40	96.3
VSS	160	75 - 280	6	5 - 29	96.3

173 and 6 mg/L, respectively. The percent removal of BOD and TSS exceeded 95 percent. Thus according to this test data the Model DF40-M receiving an average flow of about 400 gpd was capable of providing average effluent concentrations of BOD_5 and TSS of less than 10 mg/L. The NSF data did not include nitrogen species information.

Data for nitrogen species was requested from the manufacturer. The manufacturer provided some data for ammonia and nitrate. The ammonia data shown in Table 11.2 was developed by NSF at the request of the manufacturer and was performed under the test conditions of NSF Standard 40. This data indicated that with an average influent ammonia concentration of 15.9 mg/L, the Whitewater Model DF40-M could produce average ammonia effluent concentrations of 2.1 mg/L. At an average daily flow of 400 gallons, ammonia removal ranged from 77 to 91 percent. The company also supplied data for nitrate nitrogen that was generated internally with water quality analysis performed by a certified laboratory (Delta Manual no date). This data was not conducted on the same test system that was used in the NSF testing, but was based on samples collected from 10 installed Whitewater systems. Model numbers for each system tested, actual flowrates at the time of sampling, and the influent nitrate or total nitrogen concentrations were not provided. The data is of little value but indicated that a mean nitrate concentration from these various unspecified systems of 1.35 mg/L could be achieved.

Table 11.2 Ammonia Performance Data for the Whitewater Treatment System.

Sample Date	**Influent NH_3**	Effluent NH_3	Percent Removal
3/26/93	13	2.80	78.5
3/31/93	14	1.40	90.0
4/8/93	12	0.98	91.8
4/14/93	11	1.90	82.7
4/21/93	13	1.60	87.7
4/28/93	14	1.50	89.3
5/5/93	11	2.30	79.1
5/12/93	20	4.60	77.0
5/21/93	26	2.20	91.5
5/26/93	25	2.10	91.6

The NSF data also indicated that a mixed liquor total suspended solids (MLTSS) of about 3,444 and a mixed liquor volatile suspended solids (MLVSS) of 2,538 mg/L was maintained in the aeration basin. Additionally the DO concentration in the basin averaged 1.4 mg/L with an effluent concentration of 3.1 mg/L. This indicated that oxygen was being delivered to the system and that levels adequate to maintain aerobic conditions were met. A review of other literature sources did not provide any additional details about this system.

11.7 Field Trial Results

11.7.1 Flow Characterization

A detailed flow characterization study was conducted for this site from June 11 - 25, (Julian day 161 to 175) 1998. This study provided information about the flow patterns from household activities based on hour to hour and day to day variations. The overall frequency of flow events as indicated by the number of pump cycles is shown in Figure 11.3. This pattern reveals that most flow events for this household occurred between 5:00 AM and 11:00 PM. This is more clearly shown in the mean hourly hydraulic load profile for this site shown in Figure 11.4. This shows wastewater generated from the site was highly variable at any given hour in the day. For many hourly periods the standard deviation exceeded the mean hour usage. In addition, this data indicates that our sampling program which used a composite of flows collected from 4:00 PM to about 9:00 AM captured about 66 percent of the flow based on time. Sampling did not collect a number of low events that occurred during the mid-part of the day. The time required to accomplish this collection could be not be incorporated into the study.

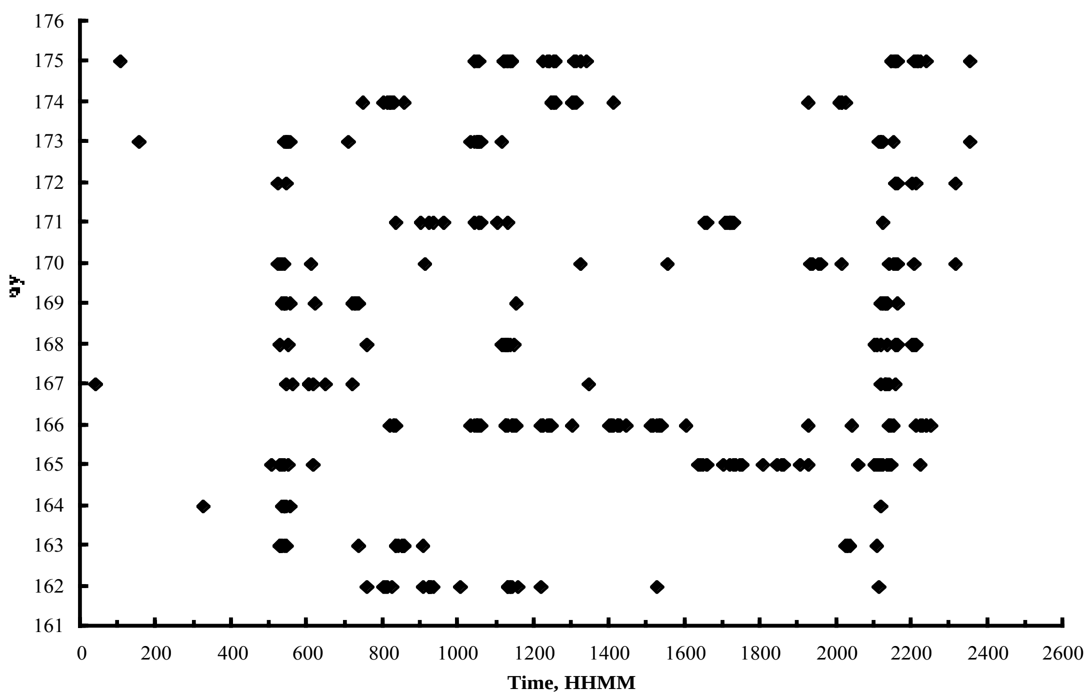

Figure 11.3. Frequency of Pump Events Recorded During the Flow Characterization Study.

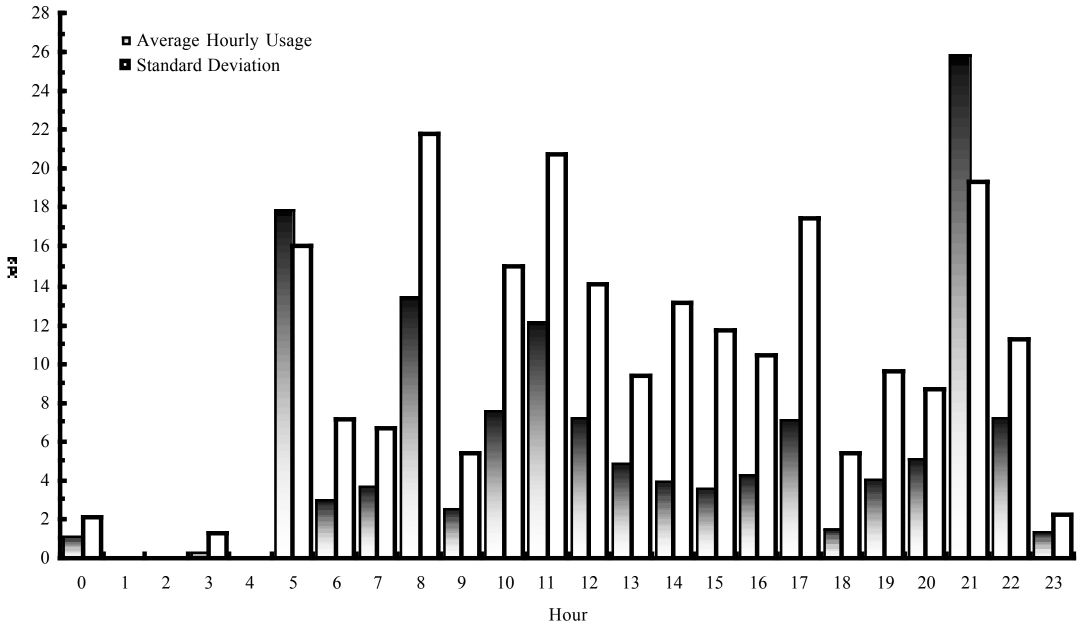

Figure 11.4. Mean and Standard Deviation of Hourly Flows Recorded During the Flow Characterization Study.

Mean flows (Figure 11.5) determined for days of the week for the flow characterization study indicates that mean flows varied from very small amounts (39.3 to 45.5 gallons) to very large amounts (233 to 302.3 gallons). No pattern of high low usage from day to day was observed in this household. The mean daily flow recorded for this period was 134.3 gpd.

11.7.2 Hydraulic Analysis

Reactor flow patterns can be characterized as either ideal flow or non-ideal flow. Two widely used ideal flow patterns used in reactor design and analysis are the complete mix flow and plug-flow (no axial mixing and uniform velocity in the direction of flow). The complete mix flow and plug flow reactors represent the two extreme mixing states. Few systems are represented by either of these flow patterns, however, many designs closely approximate these ideals. Most systems fall somewhere in between complete mix and plug flow, which is termed non-ideal flow. Non-ideal flow can be described by two basic models, the plug flow with dispersion model and the tanks in series model. These two models are roughly equivalent.

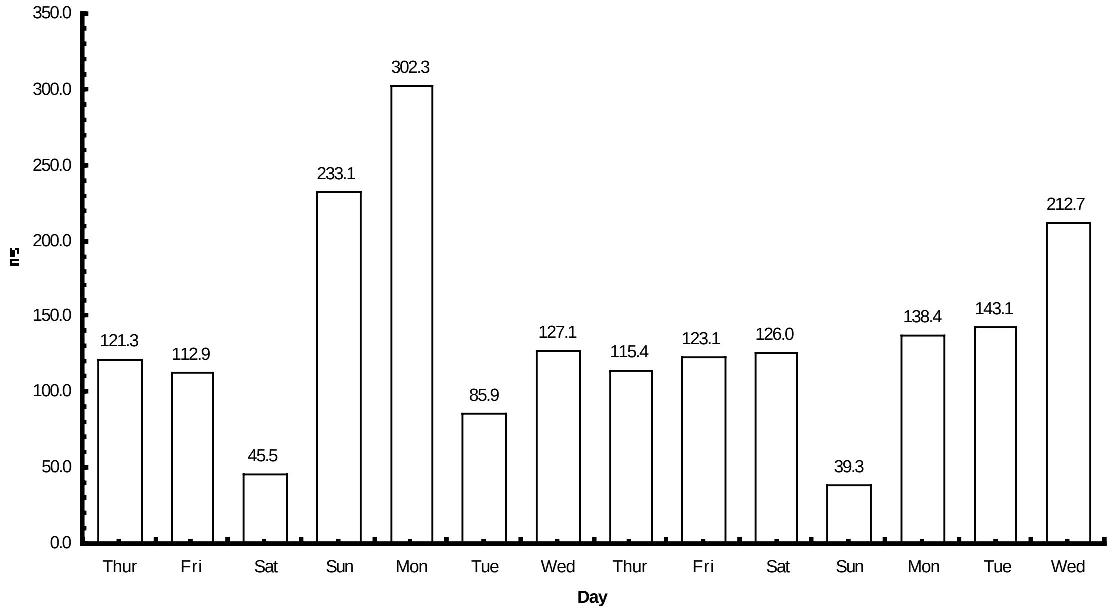

Figure 11.5. Mean Daily Flows for the Flow Characterization Study.

In order to predict what a reactor can do in terms of conversion, it is important to determine which flow pattern best represents that particular system. The flow pattern for a particular system can be analyzed by determining the residence time distribution (RTD) of material flowing through the vessel.

The RTD for a particular reactor vessel can be found by introducing a non-reactive tracer into the vessel and measuring its concentration in the vessel outlet vs. time (response curve). The tracer addition can be conducted by either a pulse or step injection. The pulse injection was used for all tracer studies discussed in this report. The pulse injection is characterized by instantaneously introducing a known mass of tracer into the fluid entering the vessel.

The tracer study for this system was conducted from June 11 to June 25, 1998. On June 11, 1998, 454.6 grams of bromide (Br) were introduced into the Whitewater system as a sodium bromide solution by the methods outlined in Chapter 8 of this report. During the tracer study, the concentration (C) of Br on the system effluent was measured and recorded as a function of elapsed time in hours from the beginning of the input of tracer into the system. This concentration vs. time (C vs. t) data was used directly and in conjunction with flow models to predict actual system behavior in terms of detention time and flow patterns. The C vs. t data for the Whitewater system was used to construct a C vs. t curve, which is shown in Figure 11.6.

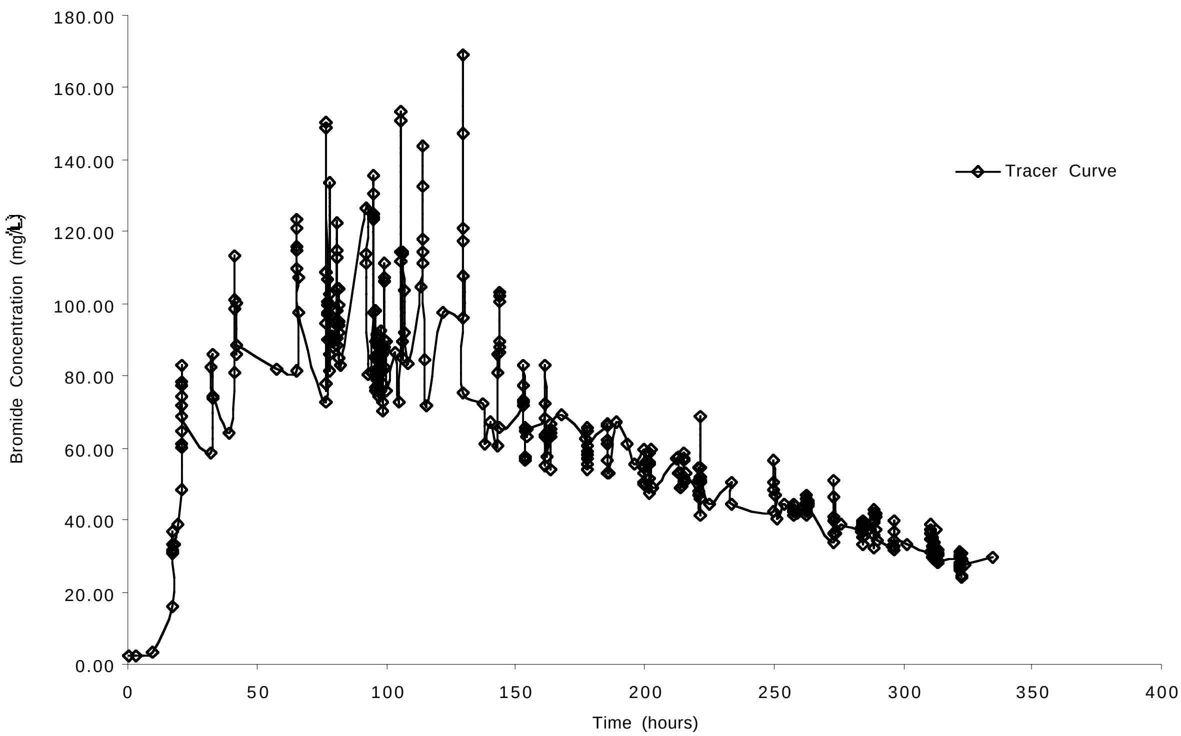

Figure 11.6. Whitewater System Tracer Study Concentration vs. Time Curve.

Direct observation of the shape of the C vs. t curve for the Whitewater system shows no evidence of short-circuiting in the system, which would be demonstrated by a sharp and early peak.

The mean and variance of a tracer curve are directly related to the system detention time and are the two quantities for describing tracer curves that are used in all areas of tracer experimentation. As previously mentioned in the Methods, the mean is the mean detention time of the system, while the variance tells how spread out in time the curve is. Using equation 8.3 (Chapter 8) the calculated mean detention time for the Whitewater system is as follows:

Mean = $\sum(t_{i+1} + t_i)(C_{i+1} + C_I)(t_{i+1} - t_i) / 2\sum(C_{i+1} + C_i)(t_{i+1} - t_i)$

= 147.46 Hours

The variance was calculated using equation 8.4 and is as follows:

$$\text{Variance} = (\sum (t_i + t_{i+1})^2 (C_i + C_{i+1}) (t_{i+1} - t_i) / 4\sum (C_i + C_{i+1}) (t_{i+1} - t_i)) - \text{mean}^2$$

$$= 6{,}981.61 \text{ hours}^2$$

The calculated mean detention time 147.46 hours, along with the C vs. t data was used to construct the E curve for the system using the methods outlined by Levenspiel (1993) and which were discussed in general in Chapter 8. Namely, $E_t = C_i$/area, and $E_\theta = E_t$ *(mean) and θ = t/mean. The E curve for the Whitewater system is shown in Figure 11.7. To determine which flow pattern approximated the Whitewater system, the E curve was compared to theoretical E curves, such as the ones shown in Figure 11.8. From this comparison, it was determined that the Whitewater system E curve did not approximate the ideal mixed flow region or the ideal plug flow region. Therefore, the Whitewater E curve fell somewhere in between the two extremes of complete mix flow and plug flow, namely, the intermediate region that can be modeled by plug flow with dispersion or complete mix reactors in series.

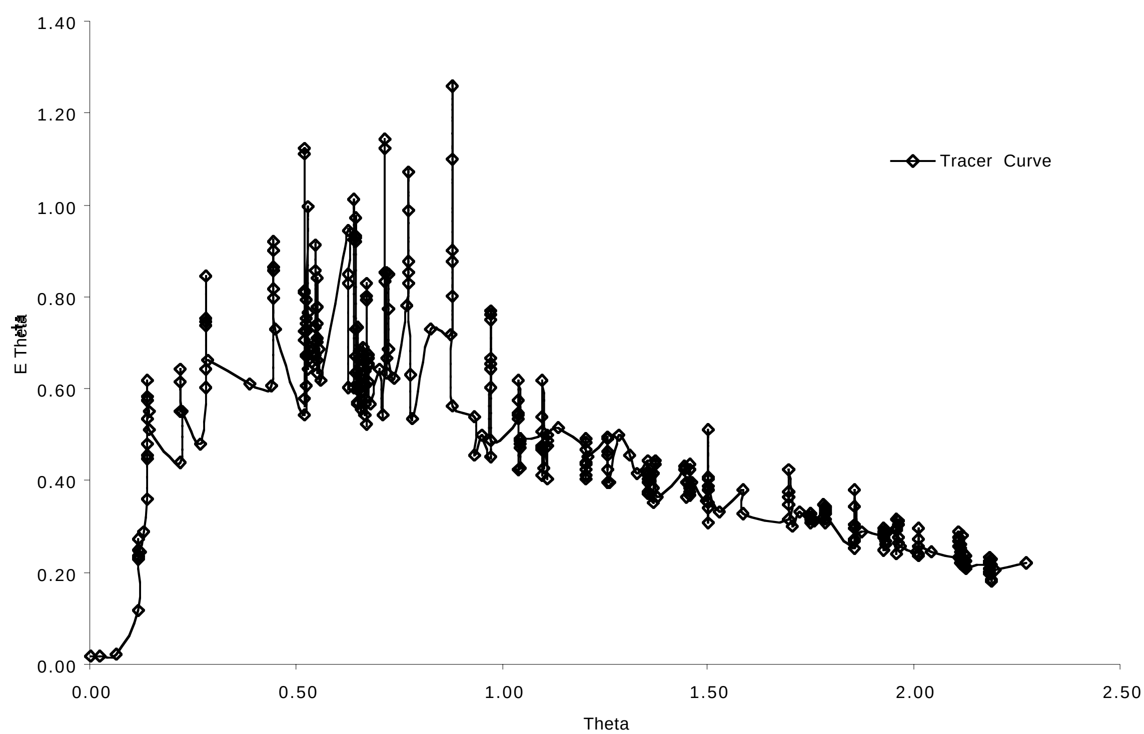

Figure 11.7. Whitewater System E(θ) Curve for the Tracer Study.

The theoretical E curves (Levenspiel, 1993) shown on Figure 11.8, were developed for different vessel dispersion numbers (D/uL values). The dispersion number (D/uL) is a dimensionless group which measures the extent of axial dispersion. Thus, as D/uL approaches 0 in equation 11.1 (Levenspiel, 1962, 1972), axial dispersion is negligible, hence plug flow and as D/uL approaches infinity in equation 11.1, axial dispersion is large, hence mix flow.

$$\delta C/\delta\theta = (D/uL)\ \delta^2 C/\delta z^2 - \delta C/\delta z \qquad (11.1)$$

where,

θ = t/mean

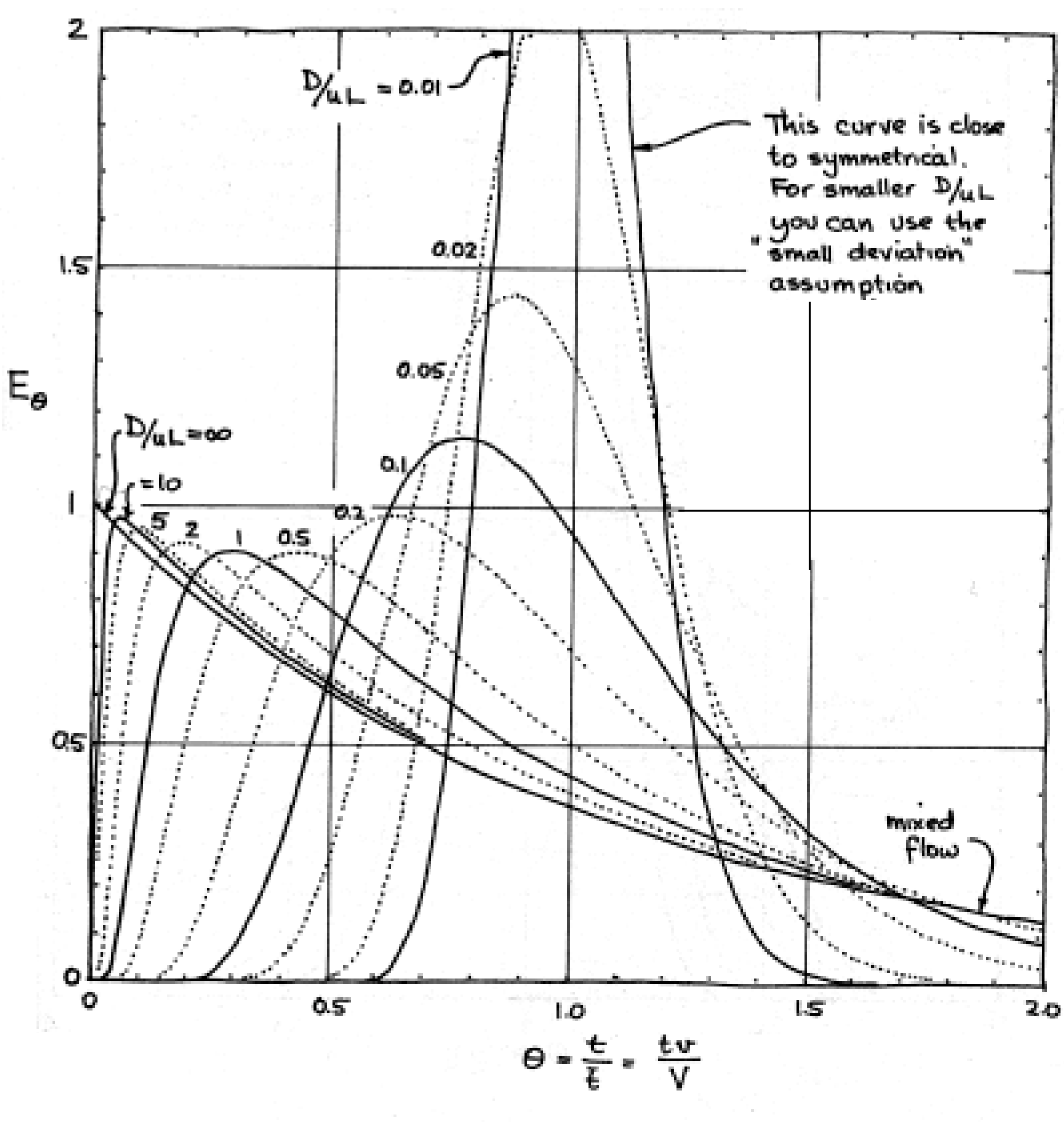

Figure 11.8. Theoretical E Curves for Various Dispersion Coefficients, Levenspiel (1993).

Levenspiel (1962) developed expressions relating the dispersion number (D/uL) to the mean and variance of the C vs. t tracer curve data. However, these expressions are dependent on the deviation from plug flow. For example, for D/uL< 0.01, the deviation from plug flow is small, whereas, for D/uL>0.01, the deviation from plug flow is large. In addition, for large D/uL, what happens right at the entrance and exit of the vessel strongly affects the shape of the tracer curve as well as the relationship between the parameters of the curve and D/uL. For large D/uL, the measured curve is unsymmetrical with a somewhat extended tail. In this situation the flow conditions at the injection and measurement point (called the boundary conditions) will influence the shape of the obtained C curve. Two possible cases that have been treated by workers in this field are the closed vessel and open vessel. For the tracer studies in this report, the closed vessel situation was represented, since the tracer entered and left the vessel in small pipes. The theoretical E curves shown in Figure 11.8 were constructed for the closed vessel situation by numerical methods using the following equation in terms of the variance and the mean of the tracer curve:

$$\text{Variance/mean}^2 = 2\ D/uL - 2(D/uL)^2\ (1 - e^{-uL/D}) \qquad (11.2)$$

The next step in the Whitewater system hydraulic analysis was to simulate the system using the Impulse program to determine how well the actual system approximated the plug flow with dispersion model. The Impulse program was used to simulate the Whitewater system as a plug flow reactor with dispersion. The Impulse program utilizes equation 11.2 to determine the theoretical best fit curve to the experimental C vs. t data by running the program in the regression mode with regressable parameters or variables, such as inlet flowrates, reactor volume, dispersion number and inlet concentrations. The fit of the experimental data is based on equating the variances of the two curves about the center of gravity (mean residence time) of the distribution (Weber, 1972). The best fit of the simulated curve vs. the experimental (tracer) curve was determined by visual inspection from the plotted Impulse output of C vs. t data.

As discussed in Chapter 8, two scenarios, each of the plug flow with dispersion and complete mix flow models, a total of four, were used to simulate the Whitewater system. The first scenario for the plug flow with dispersion model, consisted of holding the reactor vessel volume constant and allowing Impulse to calculate the reactor influent flowrate and concentration, while insuring that the amount of bromide tracer remained at the actual amount of 454.6 grams. Figure 11.9 shows the tracer curve vs. the simulated curve using this scenario. The Impulse program also calculated the dispersion number (D/uL) as 1.15. While the curve fit is excellent, the program output of calculated variables did not match the actual system. For example, the calculated dispersion number using equation 11.2 by a trial and error procedure was as follows:

$$\text{Variance/mean}^2 = 6{,}981.61/(147.46)2 = 0.321 \qquad (11.2)$$

$$\text{Variance/mean}^2 = 0.321 = 2\ D/uL - 2(D/uL)^2\ (1 - e^{-uL/D})$$

By trial and error D/uL = 0.20, which is within the plug flow with dispersion range. However, the Impulse calculated dispersion number of 1.15 is highly skeptical since for values of D/uL>1 the assumption of plug flow with dispersion should not be used (Levenspiel, 1993).

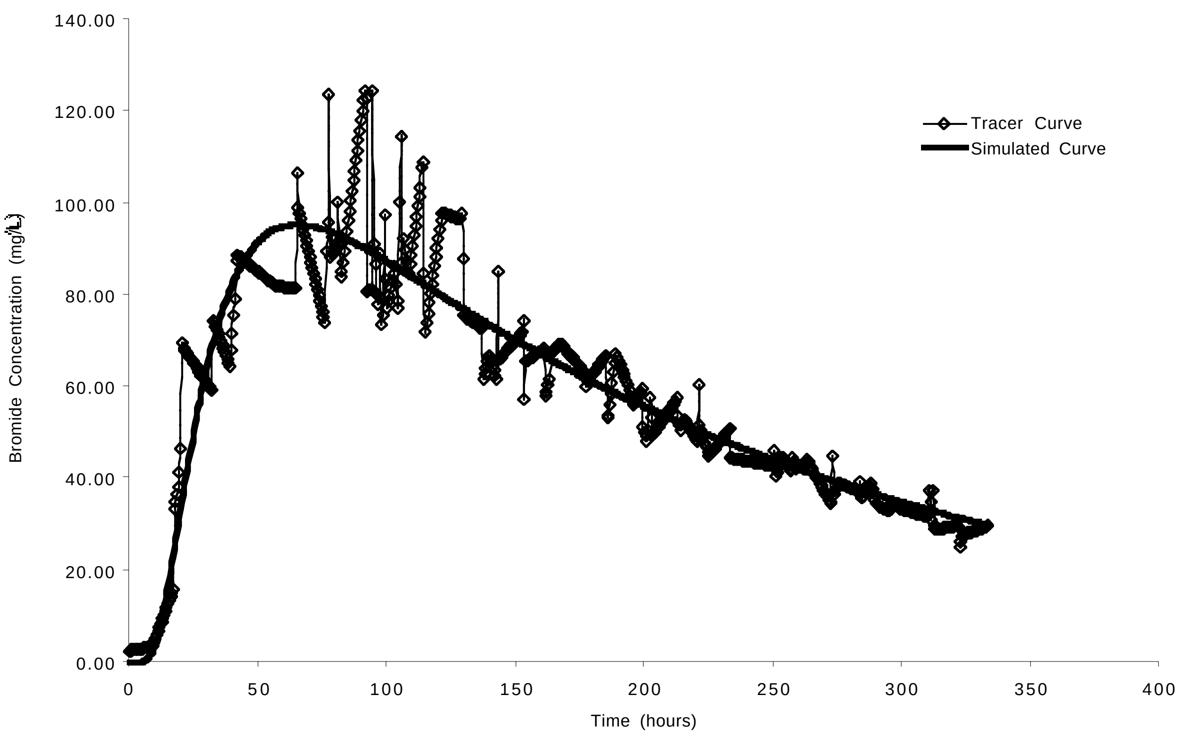

Figure 11.9. Tracer Curve vs. Plug Flow w/Dispersion Simulated Curve for Whitewater System - Calculated D/uL = 1.15.

Also, the Impulse calculated inlet flowrate of 8.63 liters/hour is considerably less than the actual average flowrate during the tracer study of 21.91 liters/hour (138.9 gallons/day). This value was calculated by dividing the total volume pumped by the total time. Furthermore, a reactor volume of 2,120 liters (560 gallons) was used vs. the actual reactor volume of 3,440 liters (909 gallons) in order to maintain the mass of bromide tracer at the actual value of 454.6 grams.

The second modeling scenario for the plug flow with dispersion model, was carried out by holding the inlet flowrate and concentration constant, with the flowrate at 21.91 liters/hour (actual avg. flowrate) and the concentration necessary to insure a pulse input of 454.6 grams of bromide tracer into the system. The reactor vessel volume was varied by Impulse. Figure 11.10 shows a plot of the simulated curve vs. the tracer curve constructed by using the output of C vs. t data from Impulse. Although, the curve fit seems worse than the first scenario, the Impulse calculated dispersion number (D/uL) for this scenario was 0.26, which is comparable to the calculated value of 0.20 using the actual C vs. t tracer data. Furthermore, the influent flowrate is

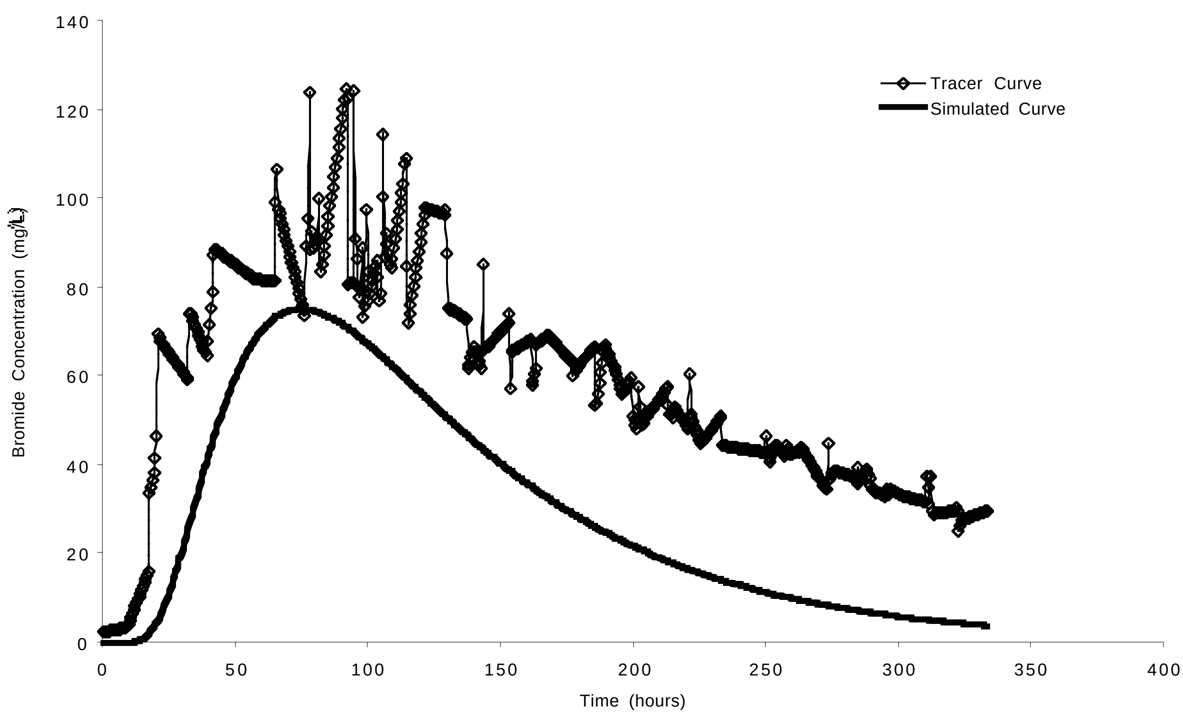

Figure 11.10. Tracer Curve vs. Plug Flow w/Dispersion Simulated Curve for Whitewater System - Calculated D/uL = 0.26.

the actual average flowrate already calculated as 21.91 liters/hour (138.9 gallons/day). The reactor vessel volume calculated by Impulse was 2,830 liters (748 gallons), which is comparable to the actual reactor vessel volume of 3,440 liters (909 gallons).

The only concern with the second scenario is that the simulated curve seems to have a good fit to the experimental curve at the beginning of the experiment but seems to diverge towards the end of the experiment. This may be caused by experimental error in calculating the bromide concentration. The Cole Palmer Bromide Electrode model 27502-04 seemed to show a shift in calibration curve as a function of time elapsed from the beginning of the experiment. This shift was found to be attributed to a deposition of a certain type of material on the probe during the course of the two-week experiment. It is speculated that due to the experimental error introduced by material deposition on the Br probe, the measured Br concentrations were higher than actual, thus causing the tail-end of the tracer curve to not drop closer to zero towards the end of the two-week experiment. The simulated curve does approach zero as it should as demonstrated in Figure 11.10. Based on the results of the Impulse simulation of the Whitewater system as a plug flow with dispersion reactor, the dispersion numbers (D/uL) of 0.2 (C vs. t experimental data) and the value of 0.26 calculated by Impulse show that the deviation from plug flow is large (D/uL>0.01). Therefore, as D/uL approaches infinity, the system approaches complete mix flow behavior. While the Whitewater system exhibits Plug Flow with dispersion, a simulation of the system as a complete mix flow reactor was performed using Impulse for illustration purposes. Once again, two different scenarios were used for the complete mix flow simulations: 1) vary inlet flowrate, and 2) vary reactor vessel volume. Figures 11.11 (vary flowrate) and 11.12 (vary volume) show the tracer curve vs. the simulated curve for the two scenarios.

The inputs to the Impulse program for the simulation of the Whitewater system as complete mix flow for the two scenarios were: 1) a constant reactor vessel volume of 2,910 liters (769 gallons) to satisfy actual pulse tracer input of 454.6 grams, and 2) constant flowrate of 21.91 liters/hour (138.9 gallons/day) and a constant inlet concentration to satisfy the pulse tracer input of 454.5 grams. The Impulse program output for the first scenario (vary flowrate) was the C vs. t data the used to construct the simulated curve shown in Figure 11.11, and a calculated

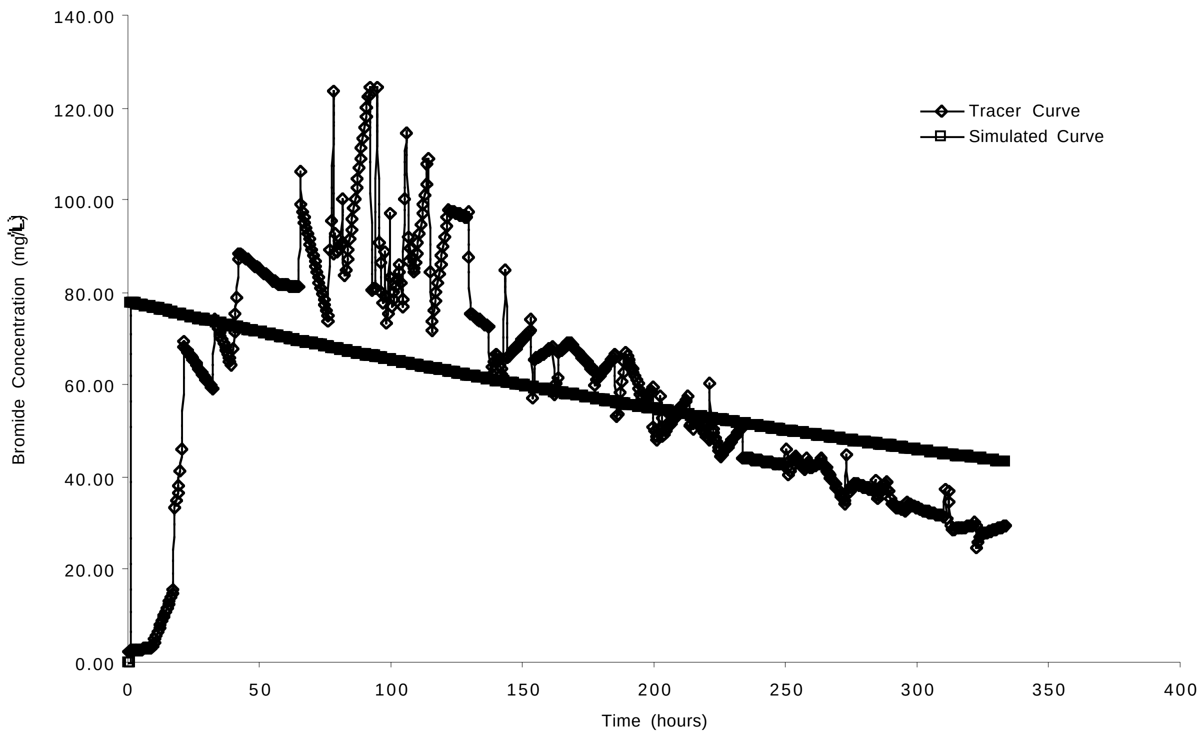

Figure 11.11. Tracer Curve vs. Mixed Flow Simulated Curve for Whitewater System - Vary Inlet Flowrate.

inlet flowrate of 5.1 liters/hour (32.34 gallons/day). The Impulse program output for the second scenario (vary volume) was the C vs. t data used to construct the simulated curve shown in Figure 11.12, and a reactor vessel volume of 3,441 liters (909 gallons). As exhibited by the plug flow with dispersion simulations, the second scenario (vary volume) shows a worse curve fit to the experimental data than the first scenario (vary volume). Again, this is suspected to be due to the experimental error in measuring the bromide concentration.

To conclude this section, the observed curve fit and comparison of calculated flowrates and volumes shows that the Whitewater system exhibits plug flow with dispersion flow, but approximates the complete mix flow extreme rather than the plug flow extreme, which is typical of this type of system (aeration system). A summary of the Impulse simulation results as well as the values obtained from actual tracer data and actual equipment characteristics are shown in Table 11.3.

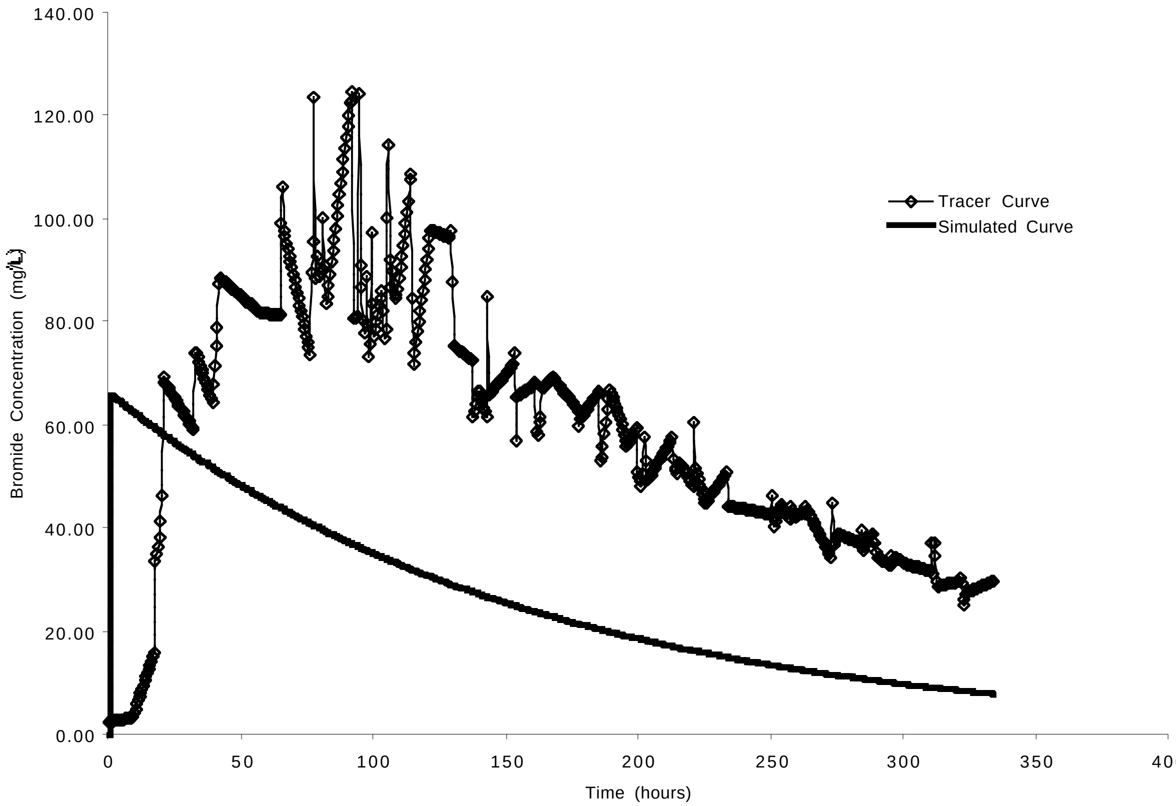

Figure 11.12. Tracer Curve vs. Mixed Flow Simulated Curve for Whitewater System - Vary Reactor Vessel Volume.

Table 11.3 Summary of Impulse Simulation Results for Whitewater System.

Scenario	Flow in (gpd)	Mass of Br Added (g)	Reactor Vol. (gallons)	Dispersion # D/uL	Curve Fit
Actual	138.9	454.6	909	0.20	N/A
PF w/Disp. (vary flow)	54.72	454.6	560	1.15	Excellent
PF w/Disp. (vary volume)	138.9	454.6	748	0.29	Fair
CMF (vary flow)	32.34	454.6	769	N/A	Fair
CMF (vary volume)	138.9	454.6	909	N/A	Poor

11.7.3 Water Quality Analysis

The Whitewater system was performance tested for 79 weeks from the September 2, 1997 through January 7, 1999 with 32 influent and effluent samples collected. The average daily flow recorded by the onsite flow meter indicated as shown in Figure 11.13, that flow varied between 86 and 202 gpd over this period with an overall average flow of 161.8 gpd. This averaged flow over the test period was consistent and steady and showed no major differing patterns. The flow was recorded on the effluent side of the treatment unit and thus some damping of extreme flow events could be expected, as the treatment unit provided some flow equalization. The calculated per capita generation of wastewater assuming three occupants in the household was 53.9 gpd/c/d. this is slightly higher but similar to values of 45 to 50 reported in other studies (USEPA 1980). It is higher than values reported by Huang (1997) on other onsite systems in Bernalillo County.

A summary of the design and operational parameters for the Whitewater system are shown in Table 11.4. At design flow the unit had a hydraulic detention time of 1.81 days. The mean flow measured over the study period indicated that the unit was operating at a detention time of 5.6 days. The flows measured for the various studies showed some variation but were similar. This again indicated that the system was not hydraulically overloaded during the study period.

Temperature data (Figure11.13) for the site indicated no significant difference between the influent and effluent with mean values of 20.8 and 19.5 °C, respectively. Effluent temperature, reflecting the actual operating temperature of the process, varied from 8 to 27 °C over the study period. The actual temperature measured was affected to some degree by the residence time of the sample in the sumps. It is probable that the temperatures indicated are slightly lower than the actual operating temperatures in the system. These observed system temperatures can affect the performance of biological treatment process particularly nitrification. Boon et al., (1997) suggest that in the 5 to 25 °C range nitrification rates will decrease or increase 50 percent for every 10 °C change in temperature. Thus it is possible for this system that temperature impacted nitrification rates.

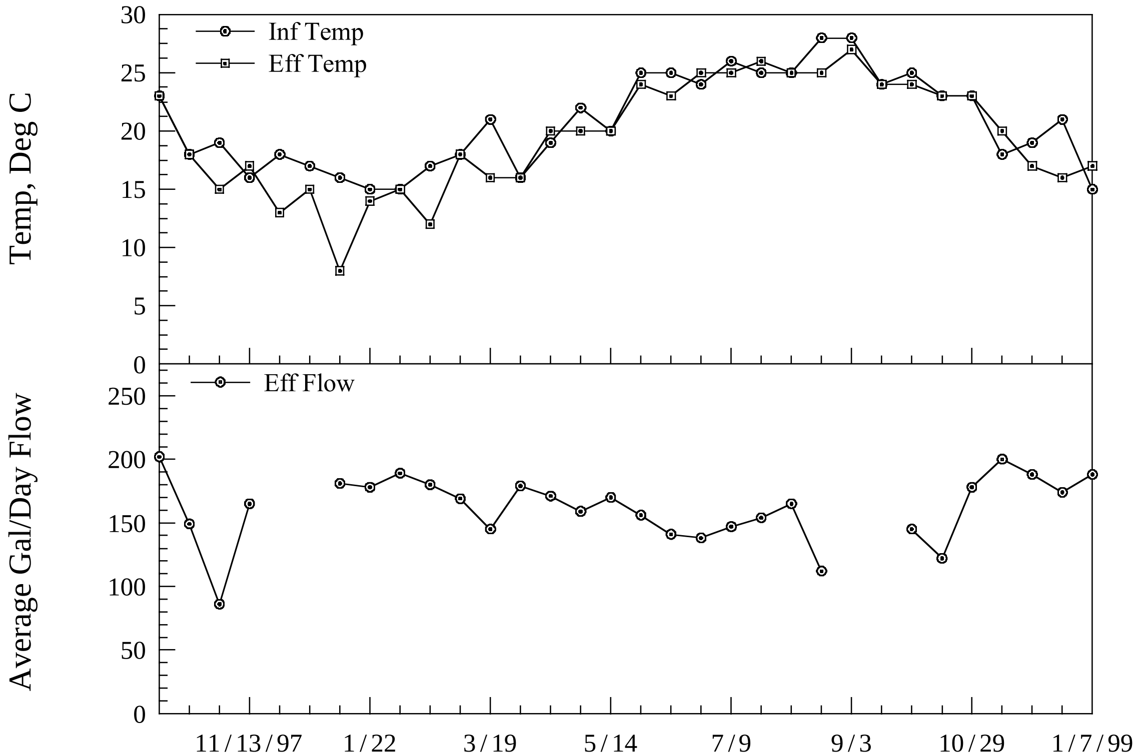

Figure 11.13. Whitewater System Experimental Data.

Table 11.4. Summary of Different Flow Conditions and Unit Operation Detention Times for the Test System.

Parameter	Design Flow	Mean Flow	Flow Study Mean	Tracer Study Mean
Flow, gpd	500	161.8	134.3	147.5
Total Unit Volume, gal	909	909	909	909
System Detention Time, days	1.81	5.61	6.76	6.14
% Difference from Design	0	67.8	73.2	70.6
Reactor Detention Time, days	1.43	4.45	5.36	4.88
Clarifier Detention Time, days	0.38	1.17	1.41	1.28
Clarifier Overflow Rate, gpd/ft^2	29.20	9.45	7.84	8.61

The electrical conductivity (EC) reflects the total dissolved solids in a particular water sample. In many cases the change in EC can be an indicator of evaporative processes or the addition of chemicals such as from a water softener, laundry operations, reverse osmosis unit, electroplating, or photo developing processes. Many of these processes may add chemicals that cannot be detected by other measurement techniques or require very specialized and expensive analysis. The EC for influent and effluent samples for the Whitewater test site are shown in Figure 11.14. The influent and effluent did not vary significantly with mean values of 1,073. And 1,081.9 respectively. This site has a water softener, but the unit was designed with a blowdown that was piped outside of the house to a French drain. Typically, water softeners are piped to the waste disposal unit and the plumbing code requires that this method be followed for all installations. The EC concentrations encountered did not indicate any unusual activities at this household. The influent samples showed somewhat greater variability than the effluent with no large spikes in the data noted.

Chloride data (Figure 11.14) for the influent and effluent averaged 83.17 and 79.1mg/L respectively. Chloride can be directly contributed by a water softener, but no large spikes in concentration in the influent were observed. Again the water softener at this site was not contributing to the wastewater flow. Additionally no increases in concentration were noted through the processes. Thus, if a spike occurred from some upstream process that was missed by our sampling efforts the equalized mass should have been detected as an increase in effluent concentration. The research team was familiar with this site since we observed this installation and prior conditions at the site. There was not a measurable correlation between EC and Chloride concentrations.

Sulfate data, an indicator of the use of certain chemicals within the household that can have a detrimental affect on treatment performance, exhibited very little variability in the influent. The influent and effluent sulfate concentrations averaged 216.1 and 226.4 mg/L, respectively and were not significantly different. Several very low values of sulfate (as low as 15 mg/L) were observed that corresponded to low or decreasing DO concentration in the effluent.

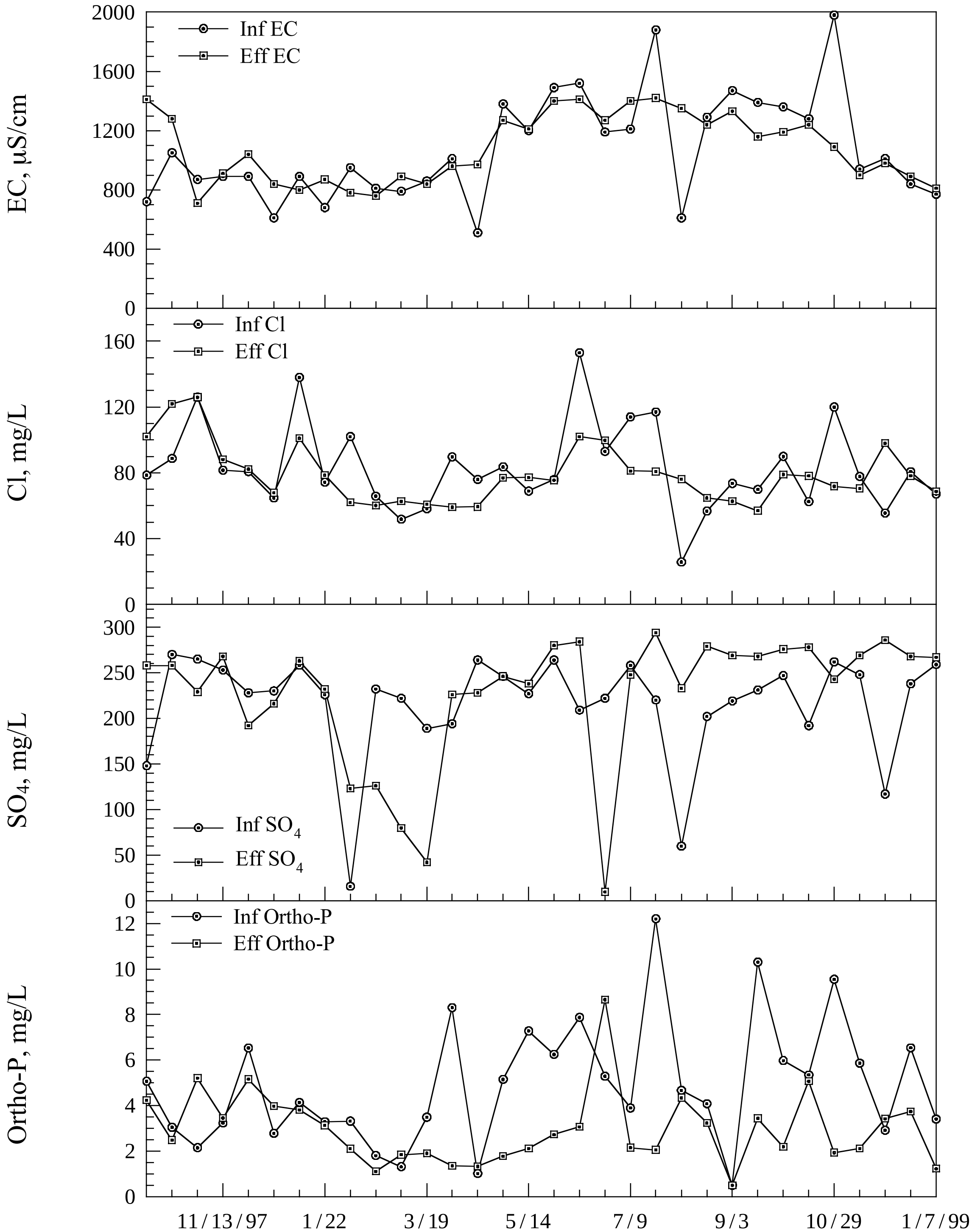

Figure 11.14. Whitewater System Experimental Data.

The removal of sulfate (SO_4) from a wastewater can be accomplished by microorganisms capable of utilizing sulfate as a terminal electron acceptor in anaerobic respiration referred to as dissimilitory sulfate reduction. The bacteria *Desulfovibrio*, *Desulfotomaculum*, and *Desulfomonas* that are obligate anaerobes reduce sulfate and produce hydrogen sulfide according to equation 11.3.

$$4H_2 + SO_4^{=} \Rightarrow H_2S + 2H_2O + 2OH^- \tag{11.3}$$

Sulfate reduction can occur over a wide range of pH, pressure, temperature, and salinity conditions, (Atlas and Bartha, 1981). Sulfate reduction can also be inhibited by the presence of oxygen, nitrate, or ferric ions. The rate of sulfate reduction is usually carbon limited and with the addition of organic compounds dissimilatory sulfate reduction rates can accelerate greatly. Evidence of sulfate reduction in an aerobic wastewater treatment process indicates that oxygen is limiting or the process is organically overloaded. This process did not appear overloaded and several instances of aerator failure were noted.

The phosphorus concentration in onsite wastewater treatment systems can impact groundwater and contribute to lake and stream eutrophication in many areas of the US. In alkaline, arid soil, phosphorus is be readily absorbed within the soil horizon and is not generally considered a major contaminant. Phosphorus removal in a treatment unit is of interest where phosphorus is a regulated pollutant. The source of phosphorus in household wastewaters can be from human wastes as well as laundry operations. Dissolved or ortho-phosphorus concentrations in the influent and effluent from this test system (Figure 11.14) averaged 4.9 and 3.0 mg/L, respectively. The influent and effluent concentrations were significantly different ($p = 0.001039$) with a system percent removal of ortho-phosphorus of 38.9 percent.

The data for pH is shown in Figure 11.15. Maintaining a near neutral pH (6 to 8) is important for the stability of biological processes. Maximum rates of nitrification will be achieved when pH is maintained in the range of 7 to 8.5 (Painter and Loveless 1983; Wong-Chong and Loehr 1978). Many cleaners and drain openers and other chemicals can drastically raise or lower pH and impact system performance. Influent pH values ranged from 6.8 to 10.0 over the course of the study while effluent values ranged from 7.7 to 9.0. The mean influent and

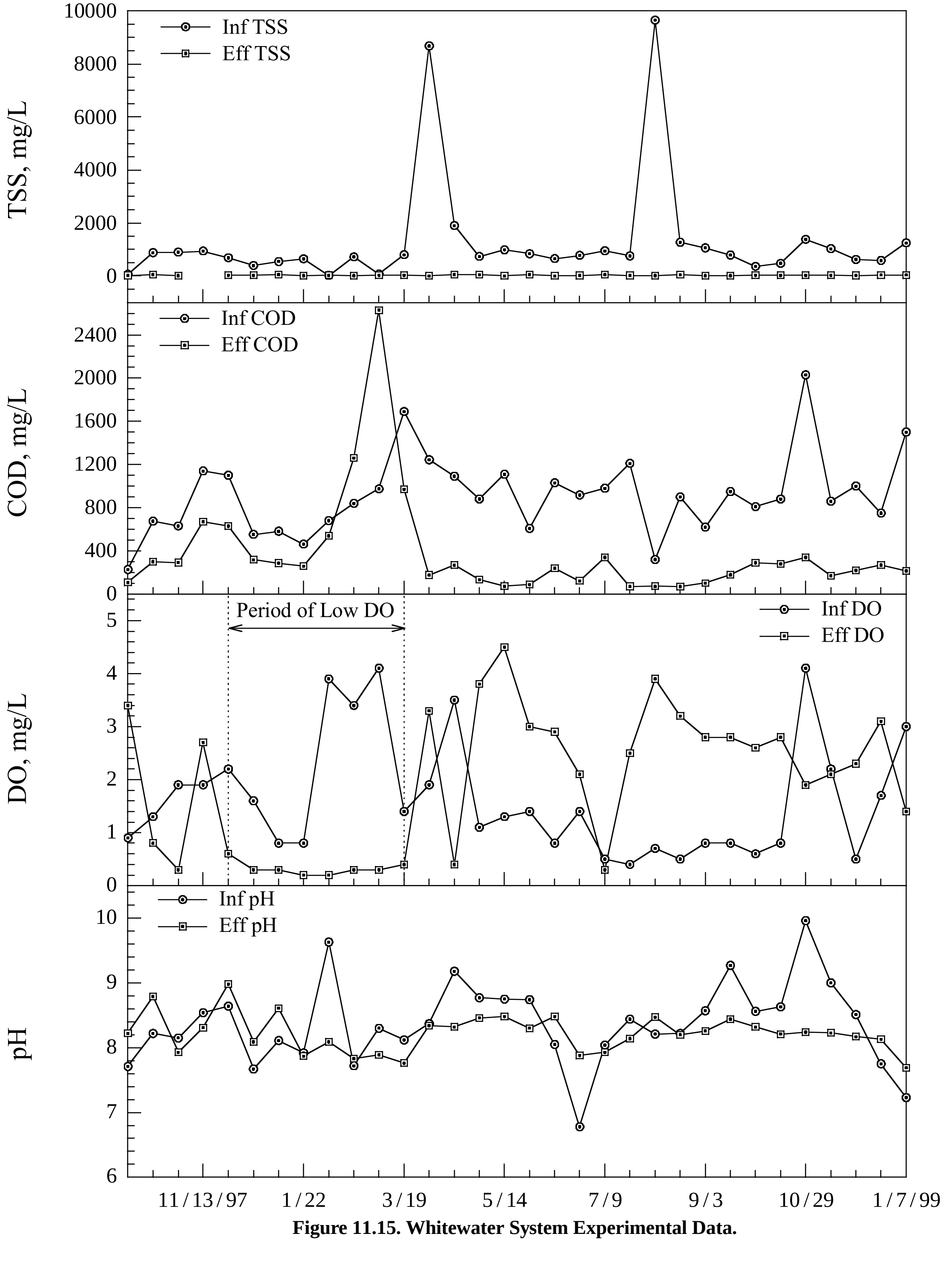

Figure 11.15. Whitewater System Experimental Data.

effluent pH values were 8.4 and 8.2, respectively and were not significantly different. While some extreme values were encountered in the influent, the effluent appeared to be much less variable and certainly within range to maintain good biological treatment.

The TSS data values for the system shown in Figure 11.15 indicated some influent events that elevated TSS concentrations over 8,000 mg/L. The average influent concentrations were determined to be 1,295.7 mg/L with a standard deviation of 2,102. The effluent values averaged 26.2 mg/L with a standard deviation of 15.4. The influent and effluent were significantly different at a p value = 0.001347. The calculated percent removal TSS was 98 percent for this system. This indicated excellent removal of TSS consistently below 30 mg/L, but not attaining a 20 mg/L standard or less.

The BOD_5 values for this system were not graphed, in part because only half the values were provided by the laboratory after samples had been submitted. Many of these samples were reported as over range or under range and were not usable. The BOD_5 values for the influent ranged from 27 to 690 mg/L with a mean of 329.6 mg/L. The BOD_5 values for the effluent ranged from 8 to 184 mg/L with a mean of 93.5 mg/L. The influent and effluent were significantly different at a p value = 0.00000539. If the performance data is examined it becomes evident that the system experienced several months of poor performance due to two factors. The first is a typical problem with biological processes that requires a startup period to develop biomass and acclimate that biomass to the influent wastewater. For many systems this can take from several weeks to several months. For systems to develop capabilities of nitrification the process usually takes a minimum of three months. Nitrifiers grow very slowly and can be affected by many substances in the wastewater and low temperatures. The second factor affecting the Whitewater system during this early period of operation was the consistent failure of the aeration system. Repeatedly we found low DO in the effluent sump and finally after some weeks of sampling informed the manufacturer's representative of this fact. The situation improved with the reconnection of the airline but problems continued with the air delivery system for several months. An alarm went off indicating loss of air pressure, but the alarm was so faint that the homeowner did not hear it.

Once the system was properly aerated and sufficient time passed for acclimation, the system performance improved. During the startup time influent and effluent BOD_5 averaged 234 and 136.2 mg/L for percent removal of 42 percent. After the plant began to experience evidence of nitrification (after 3/19/98) the influent and effluent BOD_5 averaged 391 and 36.3 mg/L for a percent removal of 90.7 percent.

The COD data values for the system shown in Figure 11.15 indicated some influent events that elevated COD concentrations over 2,400 mg/L. These events did not appear to be correlated to TSS fluctuations. The average influent concentrations were determined to be 14.1 mg/L with a standard deviation of 370.8. The effluent values averaged 374.5 mg/L with a standard deviation of 488.2. The influent and effluent were significantly different at a p value = 0.00000539. The calculated percent removal of COD was 59 percent for this system. The COD also exhibited improved performance once the startup period had passed and consistent aeration was in place. Early performance (from 10/2/97 - 3/19/98) indicated an influent and effluent COD of 796.3 and 688.7 mg/L, respectively or a 13.5 percent removal. Later performance data from 4/30/98 - 1/7/99 indicated influent and effluent COD of 964.3and 181.9, respectively or about a 90.7 percent removal of COD.

The DO data values for the system shown in Figure 11.15 indicated some variation in influent DO with concentration ranging from below 0.5 mg/L to over 4.0 mg/L The average influent concentrations were determined to be 1.63 mg/L with a standard deviation of 1.13. The effluent values averaged 1.92 mg/L with a standard deviation of 1.35. The influent and effluent were not significantly different at a p value = 0.35426. DO increased through the system, but the average reported value was below the values that should be maintained (2 to 4 mg/L) as recommended the manufacturer. In addition several periods of low DO were observed and attributed to air delivery system failure. The weak link in the system was the attachment method for the large aeration line to the drop tubes. This is a slip fitting from the aerator attached to a short piece of rubber hose and then to rigid PVC. When the unit was moved or slightly jarred by lawn mowing or some other activity in the yard, this short piece would slip off the aerator and no airflow would occur. The hose clamps to hold it in-place were pinch type clamps. These were inadequate and for a very small increase in cost the manufacturer should supply conventional screw type hose clamps. Other investigators have reported that aeration device failure is a common weakness of mechanical aerated systems (USEPA 1980; Hanna et al., 1995).

The calculated BOD to COD ratio can be useful to evaluate the biodegradability of a wastewater and it is a useful parameter to include in this analysis. The BOD/COD ratio for the influent varied from 0.025 to 0.762 with an average value of 0.39. The BOD/COD ratio for the effluent varied from 0.054 to 0.710 with an average value of 0.26. These values are similar to values of 0.4 to 0.8 reported for domestic wastewater by Metcalf and Eddy Inc. (1991).

Ammonia data for the influent and effluent is shown in Figure 11.16. For the overall study period average influent concentrations were determined to be 27.5 mg/L with a standard deviation of 25.0. The effluent values averaged 13.9 mg/L with a standard deviation of 7.6. The influent and effluent were significantly different at a p value = 0.004597. The calculated percent removal of ammonia was 49.4 percent for this system. A critical factor for the successful conversion of ammonia to nitrate is the availability of DO. It was apparent that for this system, DO was a limiting factor for nitrification during some periods of the study. Startup affects nitrification to a greater degree than other biological or physical processes. The authors' experience suggests that 60 to 90 days for acclimation of nitrifiers is not unusual depending on temperature and maintenance of adequate DO levels. The ammonia data also exhibited improved performance once the startup period had passed and consistent aeration was in place. Early performance (from 10/2/97 - 3/19/98) indicated an influent and effluent NH_3-N of 9.91 and 17.77 mg/L, respectively or a 79.8 percent increase of ammonia through the system. Later performance data from 4/30/98 - 1/7/99 indicated influent and effluent of NH_3-N 38.86 and 10.89 mg/L, respectively or about a 72.0 percent removal of NH_3-N through the system. Thus after startup and with continued aeration, the system produced an effluent ammonia concentration of less than 15 mg/L.

Nitrate is normally not found in septic effluent or raw sewage because of the limited nitrification rates. Nitrate is the product of nitrification or can be added by chemicals such as nitric acid. Drinking water standards for nitrate are 10 mg/L as N. Nitrate data (NO_3 + NO_2) for the influent and effluent is shown in Figure 11.16. The average influent concentrations were 0.06 mg/L with a standard deviation of 0.04. The effluent values averaged 1.18 mg/L with a standard deviation of 2.12. The influent and effluent nitrate data were significantly different at a p value = 0.004093. Nitrate in the effluent in concentrations above background is an excellent indicator of the process of nitrification. The data for this system shows that nitrate effluent concentrations began to rise about 5/28/98 and varied there after from about 0.4 to 8.0 mg/L. These levels

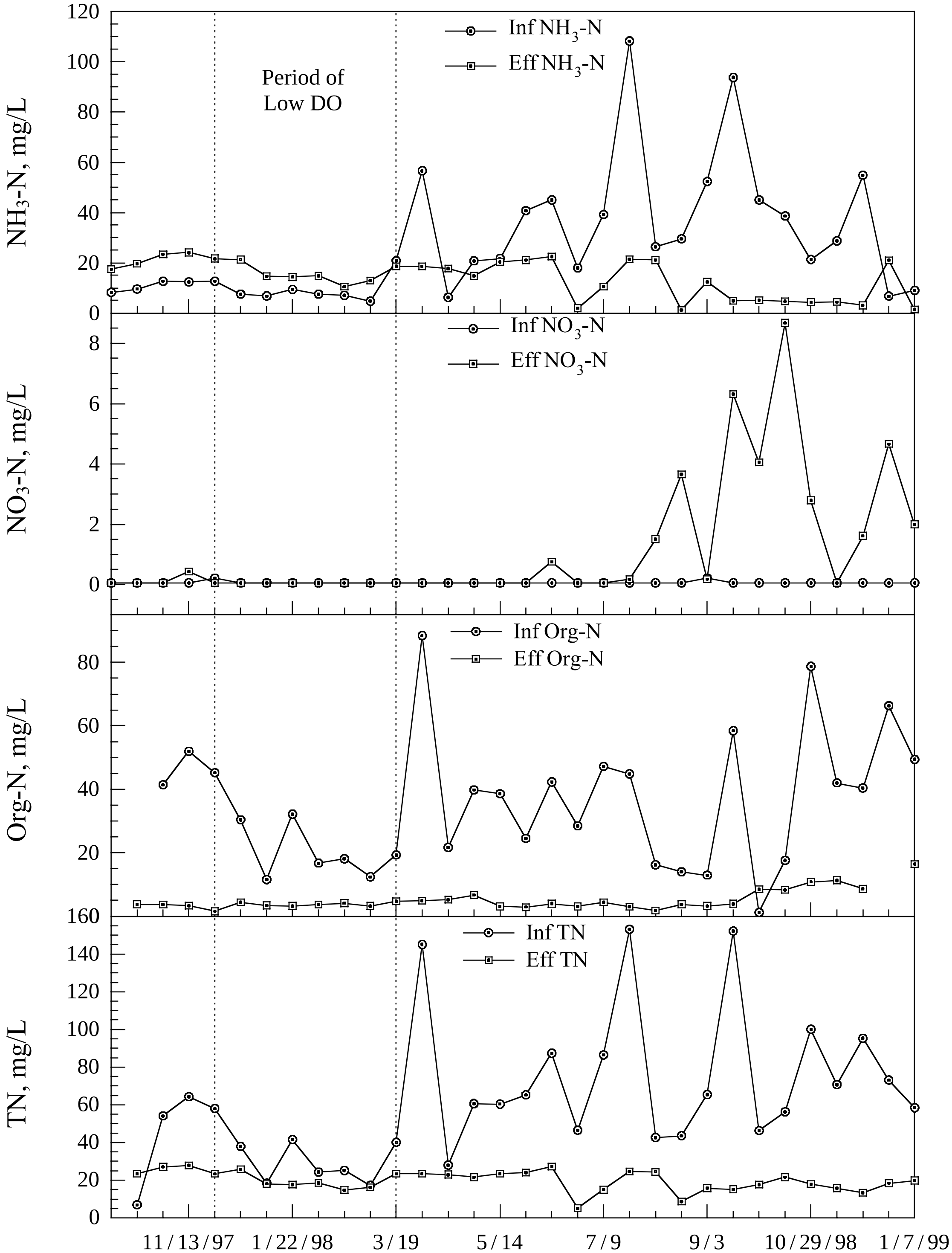

Figure 11.16. Whitewater System Experimental Data.

confirm that ammonia was being converted to nitrate, however, it is not clear that denitrification was consistent in converting nitrate to nitrogen gas. The design for this system does not include a clearly delineated area for denitrification. Raw sewage mixes in the aeration chamber and passes upward and through the clarifier. As the wastewater contacts these solids either in the aeration chamber or in the clarifier it could encounter anaerobic or anoxic conditions, but it is not clear that these conditions can be maintained. Other processes use fixed film technology to depress DO and create a controlled condition for denitrification.

Organic nitrogen for the influent and effluent is shown in Figure 11.16. The average influent and effluent concentrations were 35.1 mg/L and 5.1 mg/L, respectively. The influent and effluent were significantly different at a p value = 0.00000000107. Organic nitrogen is mostly part of the TSS and is an indicator of the solids removal of the system. Organic nitrogen is considered part of the total nitrogen and is assumed to be converted to nitrate in the disposal area of the treatment process.

Total nitrogen for the influent and effluent is shown in Figure 11.16. The average influent and effluent concentrations were 62.1 mg/L and 19.7 mg/L, respectively. The influent and effluent were significantly different at a p value = 0.0000000373. The calculated percent removal of TN was 68.3 percent for this system. The Whitewater suspended growth system could not meet a TN standard of 10 mg/L.

The fecal coliform (FC) data was highly variable with mean influent and effluent values of 6.02×10^5 and 5.25×10^4 cfu/100mL, respectively. This resulted in a removal rate of over 91 percent. Influent values ranged from 6.0×10^4 to 2.18×10^6 and effluent ranged from 0 to 2.10×10^5.

11.8 Conclusions

The Whitewater Aerobic Treatment Unit; Model DF50-FF manufactured by Delta Environmental Products, Inc. was performance tested for 65 weeks from the September 2, 1997 through January 7, 1999 with 32 influent and effluent samples collected. The flow characterization and reactor tracer analysis was also performed on the system. In addition, the installation, maintenance, and operation of the system were evaluated.

The hydraulic analysis of the system indicated no short circuiting or unusual flow problems with the system. The observed curve fit and comparison of calculated flowrates and volumes shows that the Whitewater system exhibits plug flow with dispersion flow, but approximates the complete mix flow extreme rather than the plug flow extreme, which is typical of this type of system (aeration system). The average measured flow over the study period was 161.1 gpd, which compared favorably to the flow measured during the tracer studies and flow characterization study. These flows were significantly less than the design flow (500 gpd) for the unit flow.

A summary of operating parameters for this system was presented in Table 11.5. This data indicated that the system as tested was operating well below design criteria and performance in general should be optimum. While the system was tested at an actual homesite the amount of wastewater being applied was about 68 percent below design flow. This can have both a positive and negative affect on performance. Short detention times can result in higher effluent concentrations of suspended solids due to poor settling characteristics (Laak 1986). But longer detention time can aid in the biological degradation of carbonaceous and nitrogenous components in the waste steam. A summary of the performance of the system under those operating parameters is presented in Table 11.5.

Table 11.5 Summary of Operating and Design System Parameters for Whitewater System.

Parameter	Loading lbs/day	Effluent mg/L	Percent Removal %
BOD_5	0.52	93.5	71.6
COD	1.23	374.5	59.0
TSS	1.74	26.2	98.0
NH_3-N	0.04	13.9	49.4
TN	0.08	19.7	68.3
Ortho-P	0.006	3.0	38.9

This data is a summary of the mean conditions of all the data collected for the system. TSS values were within the data extremes reported by the manufacturer, however, BOD removal for the system was somewhat lower than the manufacturer reported value of over 95 percent. Manufacturer data was not available or reliable for the rest of the data reported in the table. The critical regulatory parameters for BOD_5, TSS, TN, and FC are shown in Table 11.6 for the test system. These data indicated that average values generated for the system were well above the recommended performance standards for any of the proposed zones listed. However the study period covered startup for the system and data collected from this period skewed the performance of the system. The best data for the field trials also shown in Table 11.6 indicated better performance for values for BOD, TSS, FC, and TN.

Table 11.6 Comparison of Whitewater Data and Proposed Performance Standards.

	Field Trial Data		Performance Standards		
Parameter	Overall Mean	Best Results[1]	Zone A	Zone B	Zone C
BOD_5, mg/L	93.5	36.3	30	20	15
TSS, mg/L	26.2	8.4	30	20	15
TN, mg/L	19.7	14.2	30	20	10
FC, cfu/100mL	$5.25x10^4$	$2.7x10^4$	100	50	1
COD, mg/L	374.5	181.9	N/A	N/A	N/A
Ortho-P, mg/L	3.0		N/A	N/A	N/A

[1] Mean of data after evidence of nitrification

Observations regarding system operation and maintenance indicated that the manufacturer's supplied information was well documented and was transferred to the installer and the homeowner. However the degree of training appeared inadequate to insure proper operation of the system. Maintenance problems were encountered with the aeration system on several occasions and for extended periods during the testing period. The alarm provided for low aeration pressure was inadequate to alert the homeowner. The connection between the aeration device and the airlines was poorly designed and was disconnected by simple bumping on several occasions. This is a critical inspection step for the system inspectors for this type of system.

Chapter 12 - Submerged Surface Flow Constructed Wetlands System

12.1 Site Description

Constructed wetland systems are normally considered a custom designed system and not an “off the shelf unit”. A registered professional engineer or other professionals typically provide designs for a specific site. The test system to be discussed here is a submerged surface flow (SSF) constructed wetlands system designed to treat wastewater from a three-bedroom, domestic residence. The system is located in the North Albuquerque Acres subdivision on a 1.5 acre lot. The layout of the system is shown in Figure 12.1. Hydra Inc., a company located in the Albuquerque area, designed the system. A licensed installer from the Albuquerque area provided the installation. The homeowner had a previous arrangement with Bernalillo County for access to the site for sampling. Sample ports and electrical outlets at the site were readily available, but some modifications to the influent and effluent sump were done to allow for tracer and flow measurements to be performed. The influent sump was modified for a flow meter and bypass and isolation valving. The effluent sump was modified to include a submerged sump pump, flow meter, and isolation valving. A licensed contractor performed this work in July of 1997 and sampling began in August 1997.

12.2 System Description

The overall system is designed with a septic tank pre-treatment process that treats raw wastewater from the household. This process removes BOD, TSS, and oil and grease and provides flow equalization to the wetlands cell. A small pump station (1/2 hp pump), transfers the septic tank effluent to the wetland cell. A submerged header located in the front part of the cell and below the surface of the rock distributes the incoming wastewater evenly along the width of the cell. The water flows horizontally through the wetlands cell to the effluent collection header that is made up of a submerged pipe that collects wastewater and allows it to flow to the effluent sump.

The wetland cell appeared to be designed with a 2.4 to 2.8 day detention time and was 1.25 ft in depth. The rock varied from 3/8" to 1/2" in diameter with larger rock in the influent and effluent area to distribute flow. The wetland plants were well distributed throughout the bed and appeared to be a mixture of bulrush, (Scirpus), cattails, (Typha), and other volunteer native

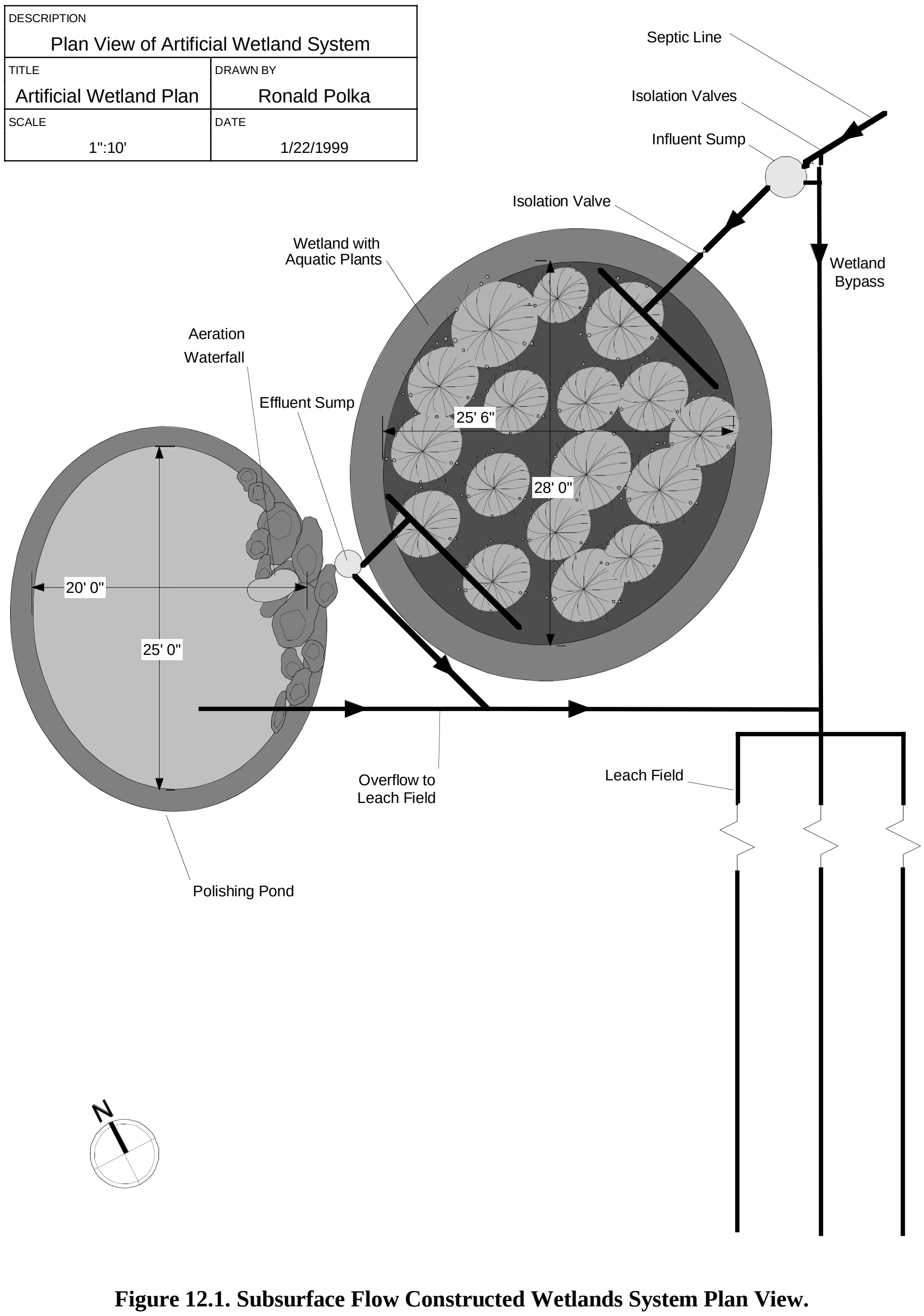

Figure 12.1. Subsurface Flow Constructed Wetlands System Plan View.

vegetation. A standpipe located in the effluent sump was used to adjust and set water levels in the wetlands. The treated wastewater flows from the sump by gravity to either a conventional drain field or to a second-stage, surface flow wetlands for water storage and further treatment. Treated wastewater from the surface flow wetlands/polishing pond could be pumped to the drain field as needed for disposal. A small recirculation pump lifted water in the surface flow wetlands, to a landscape waterfall that provided aeration and further treatment of the wastewater. This aerated wastewater was returned to the surface flow wetlands/polishing pond. The process as designed was a two-stage system, but only the first stage was tested. The design of the first stage was an older design based on BOD removal kinetics and was not optimized for nitrogen removal.

12.3 Process Description

Wetlands wastewater treatment systems or "treatment wetlands" can be either free water surface (FWS) or submerged surface flow (SSF) systems. SSF systems are the predominant type of wetlands in the Bernalillo County area. SSF constructed wetlands systems use a bed of soil, gravel or rock as a substrate or media for the growth of rooted, emergent wetland plants such as cattails (Typha) or bulrush (Scirpus). Normally, several species of plants are growing in the wetland system. Wastewater flows horizontally through the bed media contacting a mixture of aerobic, anaerobic, and facultative microbes living in association with the substrate and plant roots (Kadlec and Knight, 1996). Microbially the system is considered a fixed film process. The rock in these systems ranges in size from 6 to 150 mm (0.25 to 6 in) with 13 to 76 mm (0.5 to 2 in) typical (Reed and Brown, 1992), and a typical bed depth of 0.5 - 0.7 m (1.5 – 2.0 ft). Many systems are designed with a cap of smaller "pea gravel" to allow for easier propagation of newly introduced plants. The water level in these systems is maintained below the rock surface by an adjustable standpipe located at the outlet sump.

Length to width ratios, which affects the reactor hydraulics of the system, vary from 10:1 to 1:1 with a typical ratio of 2:1 (USEPA, 1993a). These systems require pre-treatment such as simple sedimentation (septic tanks) before the wastewater enters the wetland cell. Pretreatment is critical to prevent excessive loading of suspended solids (TSS) that might cause clogging of the interstitial rock spaces resulting in possible "ponding" or system short circuiting of the wastewater flow.

Wetlands have reportedly been effective for reducing high levels of five-day biochemical oxygen demand (BOD_5), TSS, and nitrogen, as well as significant reduction in the levels of trace metals, trace organic and pathogens in a number of applications (Reed et al., 1996). There are more than 10,000 operating systems in the U.S. covering every region and ranging in size from small home systems to large municipal facilities with daily flows of over 3 million gallons. New Mexico has over 40 operating wetland systems of various designs, most of which were constructed in the past five years.

The design approach for many SSF constructed wetlands is outlined in several reports and manuals (Kadlec and Knight 1996; USEPA 1993a; TVA, 1991;WPCF 1990). These reports generally assume a plug-flow configuration using equation 12.1 to describe the removal of BOD_5 as the major design variable. This equation incorporates the hydraulic retention time (HRT), water temperature, porosity of the rock media, and a first-order kinetic constant K_T to determine the bed size requirements. K_T is adjusted for temperature affects. The K_T value actually estimates

$$C_e/C_o = e^{(-K_T \times HRT \times ø)} \qquad (12.1)$$

where;

C_e	=	effluent BOD_5, mg/L;
C_o	=	influent BOD_5, mg/L;
Ø	=	rock porosity, %;
K_T	=	temperature dependent first order reaction rate constant, $days^{-1}$; and
HRT	=	hydraulic residence time, days.

the aerial transfer of oxygen to a 2 ft deep SSF bed for removal of BOD_5 in the system. (Metcalf and Eddy, Inc. 1991; WPCF 1990). Estimated oxygen transfer rates through the surface of SSF constructed wetlands planted with emergent plants range from 5 to 45 gm O_2/m^2-day with average values assumed to be 20 gm O_2/m^2-day (1988 USEPA). These transfer rates are assumed to be a combination of simple diffusion and active transport through the roots of the emergent plants. The K_T value assumes that all degradation of BOD_5 occur through an aerobic pathway. It is critical to this design approach that oxygen assumptions for the K_T values are not exceeded or significant shortfalls of oxygen will occur and poor system performance will result (Metcalf and Eddy, Inc. 1991).

The removal of suspended material (TSS) in SSF systems occurs rapidly in the first part of the wetland bed and is assumed to be a process of sedimentation, entrapment, and filtration (Zachritz and Fuller 1991). No equations are available for predicting or designing for the removal of suspended solids, but most systems appear to provide good removal of TSS. TSS does impart an oxygen demand to the system as the solids are trapped and the degradable fraction is oxidized. TSS also contributes to the TN of the system because the solids contain organic nitrogen that can result in the release of ammonia into the wastewater. Additionally the characteristics of the solids in the influent can be quite different from the solids discharged in the effluent.

In SSF constructed wetland systems, nitrification and denitrification are reported to be the major pathways for ammonia removal (White 1995). Plant uptake of nitrogen is estimated to be less than 20 percent of the total nitrogen removed by SSF constructed wetland systems. The process of nitrification is optimal when DO is in the range of 2 to 7 mg/L; however, some nitrification will occur at DO concentrations down to 0.3 mg/L (Reddy and Patrick, 1984). Nitrification is considered the rate-limiting step for nitrogen removal in constructed wetland systems (White 1995).

The removal of nitrate in SSF constructed wetlands appears to occur rapidly once ammonia is converted to nitrate. Gersberg et al. (1984) reported efficiencies of 97 percent for total inorganic nitrogen (ammonia and nitrate) and 94 percent for total nitrogen (TN) achieved via denitrification with Methanol. Substituting plant biomass in the form of mulch as the carbon source resulted in the removal of 95 percent total inorganic nitrogen and 89 percent TN at hydraulic loading rates of 8.4 to 12.5 cm/d. Blending primary effluent as the carbon source with the secondary effluent, resulted in removal efficiencies as high as 79 percent for total inorganic carbon and 77 percent TN while maintaining 89 percent rates for BOD_5 and TSS. Other types of carbon-donated materials such as peat have also been effective in the nitrification denitrification process (Lens et al., 1993).

Equations for the prediction of nitrogen removal in SSF systems have historically been weak and are based primarily on linear regression analyses that lack temperature correlation parameters (WPCF 1990). Recent investigators (Kadlec and Knight 1996; Reed et al., 1996) have suggested several equations, the most notable is a version similar to the plug-flow equation presented previously (Kemp and George 1997) with a K_{20} value of 0.411 for the removal of ammonia. Reed et al., (1996) has suggested a value for K_T of 0.467 for this same model. The value represents the aerial transfer of oxygen sufficient to complete the oxidation of ammonia to nitrate.

12.4 System Installation Procedures

The installation of this system was not observed. Installation guidelines for SSF constructed wetlands are provided by Tennessee Valley Authority (TVA) guidelines. These guidelines are generally for "small" SSF systems, but "small" is not strictly defined. The smallest system designed by TVA has been for 83 gpd, a one-bedroom house with limited wastewater. "Small" may be considered as systems treating flows of 20,000 gpd or less. However, a system designed to treat 100,000 gallons per day for a rural town would also be "small" compared to a system for one million gallons per day. The design features of larger systems will follow many of the guideline criteria. The guideline identifies some differences for "larger" systems. Constructed Wetlands (CW) is a relatively new technology. These guidelines may again be revised as information improves.

Pretreatment (TVA)

- Water conservation is strongly encouraged and it is considered a pre-treatment method since it can significantly reduce waste characteristics. Low-flow plumbing fixtures will help minimize wastewater flow to the CW system. For example, efficient ultra-low flush toilets using only 1.6 gallons and less per flush versus the common 5 to 6 gallon flush are available. These fixtures will pay back their cost from water bill savings and can decrease sewage flow by about one-third or more.

- Existing Facilities - Consider replacing standard flush toilets with ultra-low flush toilets. Modify other plumbing fixtures (e.g., showers, faucets) with water saving devices. Repair any leaky fixtures as soon as they are noticed. New Facilities - Require water conserving plumbing fixtures such as ultra-low flush toilets and flow restricting shower heads and faucets.

- Septic tanks will reduce suspended solids by removing coarse and heavy solids prior to the CW, install a septic tank(s) of appropriate size and design configuration.

- Obtain septic tanks from a manufacturer whose septic tank series is approved by the appropriate state/county health department. Tanks must be free of any defects. Field repairs are discouraged and generally should not be acceptable.

- Tank failures can be minimized by providing a solid foundation beneath the entire tank. This is best done by excavating about 2 inches below the final elevation of the tank and backfilling with small gravel (1/2 inch or less) or sand. The gravel or sand can be quickly leveled, precluding humps that can cause stress failure when the tank is filled with water. Set the septic tank(s) in place at the location specified by the plans, backfilling around the sides but leaving the top exposed. Fill the tank(s) to overflow and observe for 24 hours to ensure watertightness. The local health department may adjust this time so as not to hinder the installation of the constructed wetlands system. Leave the water in the tank after the testing period. Properly close the tank after testing.

- A septic tank effluent filter (with associated vault, access riser and cover, and other standard accessories) may be installed in the effluent side of the septic tank. Options include the Zabel Model A100, Orenco Models F1248 or F1260, or equivalent. A filter will further reduce solids and organic load to the CW system and assure long-term protection of the CW against septic tank upsets and poor maintenance. The filters are typically cost-effective and low maintenance.

- If the septic tank effluent must be pumped to the CW, a combined filter and pump system may be used.

Wetland Berms (TVA)

- Surround the CW cells with earthen berms or a retaining wall to retain wastewater in the treatment system and prevent surface runoff from entering the system. The top of the berm/retaining wall should be a minimum of 6 inches above the CW bed surface (top of mulch) and a minimum of 6 inches above the existing ground surface.

- Earthen Berms - Exterior slopes should be 3:l or flatter. Interior slopes may be vertical or sloped up to 2:1, determined based on existing soil characteristics, construction techniques to be used, and landscaping objectives. Plywood can be used to shape interior vertical walls.

- Retaining walls - Use instead of earthen berms to conserve space or for terracing needs. Build with concrete blocks, crossties, landscaping timbers, or other materials that are strong and durable. Line or seal retaining walls to prevent seepage.

- Cap - For small (home) systems, the top of the berms or retaining walls may be capped with 6" X 6" landscape timbers or railroad crossties to secure the liner, prevent surface runoff from entering the cell, and improve the appearance of the cell. For larger systems with an earthen berm, a minimum top width of three feet would facilitate grass cutting.

Wetland Liner (TVA)

- Install an impermeable liner inside berm/retaining wall on the bottom and sidewalls of cell. The primary purpose of the liner is to prevent exfiltration of wastewater from the cell and infiltration of groundwater into the cell. With exfiltration, a sufficient water level could not be assured for maintenance of wetland vegetation. With infiltration, retention time needed for wastewater treatment would be reduced.

- Use a type of heavy-duty synthetic 30-45 mil membrane, such as ethylene propylene diene monomer (EPDM) rubber, polyvinyl chloride, or polyethylene, or compacted clay. Use UV resistant materials. With a synthetic liner, remove all rocks, roots and debris that might puncture the liner. A 1" to 2" layer of sand or round pea gravel between the bed bottom and the liner would provide additional protection and should be required for all installations where bedrock must be excavated. Provide leakproof seal between the liner and piping which enters and exits the cell for inlet and outlet distributors. For example, use Tank adaptors or equivalent.

Wetland Substrate (TVA)

- The most common substrate is sized, washed gravel. A preferred substrate to reduce compaction is gravel with rounded surfaces such as river pea gravel. Do not use crushed limestone unless it is the only available alternative; it can compact more due to its angular shape and has a greater potential to puncture a liner. Size - For the main substrate, use A.H.D. sizes 8 through 9 (average diameter 1/4 inch and 1/8 inch, respectively). Larger sizes (e.g., A.H.D. sizes 6, 67, or 7 - 1/2 to 3/8 inch) may be used if more readily available in certain locations, but the smaller size is preferred. Also, the larger size should be used if the septic tank effluent is pumped to the CW. Influent distribution and effluent collection - In the first and last two feet of the cell(s), use 2 to 4 inch stone around the influent distributor and effluent collector pipes to reduce influent and effluent clogging potential. One foot instead of two feet may be used for very small systems (e.g., one bedroom house) with a short cell length.

- Cleanliness - The substrate should be washed to minimize fines which will plug the pore spaces of the substrate and possibly cause surface flowing.

- Substrate Surface - The substrate bed surface should be flat to facilitate water level control, vegetation planting and growth, and prevent stagnant pools. Allowable tolerances should be 0.04 feet (0.5 inch) or less at any point on the surface for small systems (1/4 acre or less), and 0.08 feet (1.0 inch) for larger systems (greater than 1/4 acre).

- Surface Mulch - For small systems, apply a 3-inch layer of mulch on top of substrate to help control potential odors, prevent reflective sun scalding of vegetation and for visual aesthetics. Mulches may include bark, pine straw, tree chips, composted leaves, etc. (Mulch is generally not used on larger systems where water levels can be temporarily raised above the gravel surface for planting and special maintenance operations).

Wetland Influent Plumbing (TVA)

- Complete necessary re-plumbing on house sanitary plumbing to connect all household wastewater to the septic tank. Use approved plumbing standards (e.g., The Southern Plumbing Code), to determine house re-plumbing needs, type of pipe, and location and spacing of cleanouts between the house and septic tank.

- Influent Distribution -Use a header pipe to provide uniform wastewater distribution. Distribution can either be on or below the surface of the substrate. Use a buried or covered distributor for smaller systems where accessibility to the wastewater needs to be controlled, such as for an individual home where children can be present. Use a surface distributor for larger flows (above about 2,000 gpd).

- Buried Distributor - This method is used for smaller gravity flow systems to limit access to the wastewater. Use 2-inch diameter pipe for homes with up to three bedrooms with typical water use. Use 3-inch diameter pipe for a four-bedroom house. Place inlet distributors at mid-depth in 2 to 4 inch stone. Drill holes 5/8" diameter, spaced 6 inches apart on top, bottom, and each side of pipe (4 rows). Small systems with pump - make the 2 to 4 inch stone depth about 3 inches deeper; place the pipe (sized to maintain a flow velocity of 2 ft/sec or higher) on top of the stone (above water level); drill orifice holes 1/8 inch diameter, spaced 6 inches apart, in series of 3 holes facing up and 1 hole facing down (to allow the pipe to drain between pump cycles); place orifice shields made of 3 inch PVC caps over each 1/8 inch orifice facing up; and cover the distributor assembly with a material such as a piece of filter fabric, fiberglass screen, or liner, and then followed by mulch.

- Surface Distributor - This method is preferred for larger systems, both gravity flow and pumping from the pre-treatment unit. Use a 3 inch minimum (sized for flow) header pipe having tees which can be swiveled to distribute the flow evenly from each tee. Tees will have lubricated slip fittings. Preferred tee spacing on the header is 4-6 foot centers for headers less than 100 feet long and 5-8 foot centers for headers between 100 feet and 200 feet long. At the end of each header will be a plug or cap which can be removed for flushing accumulated solids. The pipe needs to be firmly anchored between tees to prevent movement during adjustment of tees.

Wetland Effluent Plumbing (TVA)

- Use a header pipe 2 inch diameter, or larger, pipe, sized for flow to provide uniform wastewater collection. A 2-inch diameter pipe is adequate for homes with up to four bedrooms with typical water use. Drill holes 5/8 inches in diameter, spaced 6 inches apart on top, bottom, and each side of pipe (4 rows). Place pipe on the cell bottom in the 2 to 4 inch stone. For clean-outs, install a capped clean-out on each end of the inlet distributor and outlet collector. Locate clean-outs at the inside edge of each cell and extend above the top of the mulch. Extend the inlet distributor clean-outs at least 12 inches above the mulch to allow observation/monitoring of increase in head due to partial pipe/gravel clogging with time.

Wetland Pumping (TVA)

- Ideally, a gravity flow system is desired. However, if wastewater cannot gravity flow from house plumbing to the septic tank and CW, the septic tank effluent must be pumped.

- Select the required pump based on flow and total head, working with the pump manufacturer.

- Always include either a septic tank filter (Orenco, Zabel, or equivalent) or a second septic tank to minimize solids pumped to the CW (sections II.B and II.C). Restrict the pumping rate by a valve to prevent or minimize surface flow in the CW. Set the dosing volume so that it does not exceed one-fourth the daily design flow. Also, adjustments (increases) may be needed in the influent cross-sectional area based on instantaneous pumping rates. Surface surging, which may eventually occur during pumping, may be mitigated by reducing dosing volume, flow rates, and/or lowering cell water depth, to provide additional surge capacity.

Wetland Water Level Control (TVA)

- Water level control and adjustment is critical to establishment and survival of the plants. Roots of emergent plants must be kept wet and the plants will not survive if they are completely covered with water for extended periods. Also, if water is allowed to stand in or above the mulch, surface odor may occur.

- Install a suitable discharge structure incorporating an adjustable water level control device at the effluent end of the cell(s) using either the swiveling standpipe or collapsible tubing option.

- For a home system, the structure may consist of an 18 inch PVC pipe section, embedded vertically in a 6 inch concrete floor pad. For larger systems to accommodate larger piping, the structure may consist of a larger PVC pipe section or a concrete block box.

- The discharge pipe from the effluent collection header will enter the control structure. Place a valve on the discharge pipe from the cell into the water level control structure immediately in front of the water level control device. The valve will allow flow to be stopped if the standpipe ever needs servicing. Connect the adjustable water level control device after the valve. For smaller systems, the water level control device may be a length of flexible/collapsible tubing such as super heavy -duty sewer hose used for travel trailers. For larger systems, a swiveling standpipe is suggested. The standpipe/tubing should allow manipulation of the water level from draining the beds to 2 inches over the surface of the gravel substrate. Provide a tight fitting lid for the water level control structure to prevent escape of possible odors, keep out leaves and other objects, discourage vandalism of the valve and piping, and preclude possible safety considerations.

- Rainfall Runoff - Slope or trench the area around the CW to divert surface water away from the system.

- Safety - The entire CW area may be enclosed with a suitable fence. This is especially encouraged with larger systems where the substrate is flooded for planting and operational measures. Fences will also discourage trespassing and prevent possible sanitary problems. Also, water level control box lids may be secured and locked.

- Provide a tight fitting lid for the water level control structure to prevent escape of possible odors, keep out leaves and other objects, discourage vandalism of the valve and piping and preclude possible safety considerations.

Wetland Effluent Disposal (TVA)

- When all water cannot be eliminated in the CW cells or local health department policy requires drainfields as a precautionary measure, route excess water to a drainfield of gravel-less leach bed tubing (or equivalent) installed according to manufacturer instructions. The size of the drainfield is typically much smaller than that required following septic tanks because of the improved long-term percolation rates of the higher quality CW effluent. One successful sizing criterion is 50 feet of 8 inch gravel-less pipe per bedroom. The bottom of the gravel-less pipe typically should be at least 12 inches above bedrock, impermeable clay, or seasonally high groundwater level.

- Land application or drip irrigation - The highly treated discharge could be land applied to an area planted with landscaping plants or with wetlands or water tolerant grasses. Consult with the local health department or regulatory agency for applicable design criteria

Wetland Vegetation (TVA)

- Use plant species that grow naturally within the region.
- Select species, which have extensive vertical and lateral root growth. Preferred species include, but are not limited to: <u>Typhaceae</u> (cattail family), <u>Cyperacaea</u> (sedge family), <u>Graminear</u> (grass family), and <u>Junacaea</u> (rush family). <u>Scirpus validus</u> (softstem bulrush) has been used successfully at several municipal systems. <u>Phragmites australis</u> (giant reed) is a very good species for wastewater treatment, but is considered a "noxious" plant in some areas due to its aggressive growth.

- Sunlight - Full sunlight for most of the day during the growing season is needed for most species. For shady locations, select shade tolerant species such as ferns.

- Ornamental species - Flowering and other types of ornamental species can also be used for aesthetic attractiveness, especially around the perimeter of the cell(s). -Several species include, but are not limited to: canna lily <u>(Canna flaccida),</u> elephant ear <u>(Colocasia esculent)</u>, calla lily <u>(Zantgdeschia aethioRica),</u> various water iris <u>(Iris pseudacorus),</u> arrowhead <u>(Sagittaria latifoli</u>), arrow arum <u>(Peltandra virginic</u>), pickerelweed <u>(Pontederia cordata),</u> and sweet flag <u>(Acorua).</u>

Wetland Planting (TVA)

- Ideally, plant vegetation during spring to early summer to obtain as much growth as possible prior to winter. This reduces winter mortality. Do not plant vegetation after 2 weeks prior to early frost date.

- Adjust the water level in a single cell system or the first cell in a two-cell system to the top of the gravel substrate.

- Space plants on no less than a one-foot center grid pattern for small systems (less than 1/4 acre) and two foot centers for larger systems (greater than 1/4 acre). This planting density should provide a uniform vegetation cover in one to two growing seasons. Use plants with a 6 to 12 inch stalk above the roots; prune if necessary. Plant through the mulch so that root portion is in the water and the stalk above water. In the second cell of two-cell systems, use sprinkler to water plants until they have at least 1 foot of new growth.

- At the completion of construction activities, dress the site. Rake any ruts and bare areas made during construction to equivalent original condition.

- Minimize erosion on earthen berms by sowing a suitable cover crop (e.g., Kentucky 31 Fesque) and cover with a straw mulch.

- If wet conditions occur at the CW discharge point, plant reed canary grass or other water tolerant species as a cover crop.

12.4.1 Manufacturer's Recommendations

TVA (1991) also developed a set of maintenance guidelines for these systems that are broadly applied and are included for this section. Constructed wetland treatment systems for small wastewater flows require minimal operation and maintenance. However, some care by the owner is required to maintain an effective and attractive system. Casual observations are needed to preclude problems or minimize identified problems. The length and detail of these guidelines should not alarm the user. Potential problems are addressed that are not expected to occur unless the system is abused.

Operational Start-Up (TVA)

- Delayed Organic Loading - Preferably, plants should grow for one growing season before continuously sending wastewater to the system. This will enhance good root development throughout the substrate. Although most systems are typically placed in service as soon as they are completed, plan and conduct an extended start-up period under reduced loading conditions, if possible. Add water or wastewater to the system to maintain the water level and liquid fertilizer for good plant growth.

- Flow Distribution - For surface distribution, adjust each swivel tee on the distribution pipes to obtain equal flow from each tee. This is accomplished by trial and error. Insert a lever (short section of like size pipe) into the tee and then gently rotate the tee to the proper elevation. Set the overflow elevations from the tees so that the distributor pipes will be about half full of water.

- Water Level - Maintain water level about 1 inch above the gravel substrate surface until the plants have about 1 to 2 feet of new growth (may not be possible in an unlined cell).

- Grass Mowing - Do not mow the newly planted grass until it is at least 4 inches high. Do not cut it any lower than 3 inches until it is fully established (for at least the first two growing months after planting). Do not blow grass clipping into the wetland cells to reduce the need for weeding the cells.

- Sprinkler Use - For systems with a second wetland cell, check the water level in the second cell at least once a week. If the water level is more than two inches below the top of the gravel (or deeper than the root depth of the plants), water the cell with the sprinkler at least weekly during dry periods of the growing season for at least 2 hours, or more frequently if the plants are not growing good.

Septic Tank (TVA)

- Do not allow the septic tank to fill with solids so that solids carry over into the CW. Solids can plug the distributor pipes and the gravel in the CW. If this does occur, sewage can back up into the plumbing and surface in the CW. Also, odor and aesthetic problems can result. These can become costly and time-consuming problems to mitigate.

- Check the depth of accumulated solids in the septic tank after the first 5 years of operation, and every two to four years thereafter. When the sum of the depths of the bottom sludge and floating scum is one-third of the distance from the tank bottom to the outlet pipe, a professional septic tank pumper should clean the tank and dispose of the septage as approved by the local health department.

- In tanks with filters (Zabel, Orenco, or equivalent) the filters should be cleaned whenever the tank is pumped. This should be done by a professional septic tank pumper. Clean filter by spraying with clean water according to the manufacturer's instructions. Direct the wash water back to the septic tank. If the house plumbing becomes clogged to the extent that none of the plumbing fixtures are draining properly, the filter is one likely source of the drainage problem. It should be inspected and cleaned as necessary. If this occurs, it indicates an upset of the septic tank caused by excessive flow or disposal of harmful chemicals that should be preventable.

Water Level Control (TVA)

- Normal Operation - Maintain water level in the first cell about one inch below the gravel substrate surface at the inlet end. Adjust water level using the pipe/tubing in the water level control structure. For a swivel standpipe, gradually rotate it down to lower the level and up to raise the level. For the flexible tubing, lower or raise the top of the tubing with the notched chain and hook on the wall. To conveniently check the water level relative to the gravel surface, remove the caps of the observation standpipes at each end of the inlet distributor, or remove a small area of mulch and dig a shallow hole in the gravel (fill hole after checking). Water levels will temporarily increase with flow surges.

- Extended No Flow Periods (e.g., long vacations) - Maintain water level in the bed. Without flow, water in the cell will evaporate in hot weather and freeze during severe cold weather conditions. Both extremes will damage roots and tubers over a prolonged period. Plan to have water added to the system as needed.

- Pump Systems - Adjust the pump floats, pump outlet valve, and the water level so that the pump cycle does not result in wastewater surging above the mulch layer. Periodic adjustments may be needed as the system matures to keep the surges below the mulch or gravel.

- Maintain the pump and any alarm system according to manufacturer's specifications.

- Leaking Joints - Check adjustable standpipe or hose in the water level control structure for leaks from joints. Repair to stop any leaks. First, shut off flow using valve located in front of adjustable standpipe or hose. Open valve as soon as repair is completed.

Surface Ponding (TVA)

- If surface ponding in a wetland cell can not be controlled by water level adjustment, it may be caused by either excessive flows above the design basis or clogging of the substrate by excessive solids from the septic tank or by microbe growth due to excessive organic loads.

- Determine if solids are collecting in inlet distributor by cleaning with a homemade cleaning gig constructed with a wire and sponge. Snake the wire from one end of the distributor pipe through the other end. Wrap one end of the wire around a sponge or other material that is large enough to be compressed when pulled through the pipe. Clean the pipe by pulling sponge through the pipe several times. A large amount of solids in the pipe indicates plugging of the wetland by excessive solids discharging from the septic tank. Draining and drying the cell for a week or more may temporarily help the problem, but correction will probably require replacement of the gravel from the inlet to the point where flow reenters the gravel. Identify and implement actions to prevent the problem from recurring, such as pumping the septic tank more frequently, installing a septic tank filter or another septic tank in series, and eliminating the use of any toxic chemicals that have the potential to "upset" the septic tank.

- If cleaning of the inlet distributor reveals little or no solids buildup in the pipe, ponding is probably caused by excessive water flow that exceeds the hydraulic capacity of the substrate. Corrective actions include use of water conserving fixtures or installation of another parallel wetlands cell. Water levels may temporarily increase with flow surges. Do not make major corrections unless the water level remains above the gravel or mulch surface for an extended period.

Inlet Distributor (TVA)

- Buried Distributor - Periodically check the water level in the cleanouts on each end of the inlet distributor. If the water level in the cleanouts is obviously higher than the top of the gravel, holes in the distributor pipe or the large stone around the pipe are clogging. Clean the pipe using a homemade cleaning gig described previously. If pipe cleaning doesn't correct the problem, the large stone can be cleaned by carefully pouring oxidizing chemical such as bleach or hydrogen peroxide into the distributor pipe cleanouts. Replace any wetland plants that may be killed in the inlet area.

- Small Systems With Pump - Clean the distributor pipe once per season (spring, summer, fall, and winter) by removing the end caps and running the homemade gig described previously through the pipe several times.

- Surface Distributor - Check and maintain the distributor tees so that the flows are about equal. A tee may become partially blocked by solids, algae, or other articles. Flush solids out of the distributor pipe by temporarily removing an end cap/plug or turning an end tee down one at a time. The flow of water should flush most solids. Use a garden hose to remove remaining solids. Remove any articles such as paper, sticks, or rags that may block a tee. Periodically brush each tee to remove accumulated algae growth.

Liner (TVA)

- Maintain cover over the sides of synthetic liners (e.g., EPDM, polyethylene, PVC, hypalon, neoprene, butyl rubber, etc.) which extend above the substrate and water level to prevent UV degradation. Periodically check for liner leaks. Dyes should be used to verify suspected leaks. Drain cell, remove gravel in the suspected leak area, locate leak, and patch the liner following manufacturer's instructions. Leaks around the inlet and outlet pipes may be caused by caulking pulling away from the liner. If so, re-caulk as necessary. Draining and repairing leaks should be accomplished within one day to reduce risk of killing the wetlands vegetation.

Berms/Retaining-Walls (TVA)

- Repair any earthen berm erosion as soon as it is noted.

- Repair leaks around berms/retaining walls as soon as noted by plugging, sealing, etc.

- Mow earthen berms or around retaining dikes to maintain an attractive site.

Vegetation (TVA)

- Check the vegetation for signs of disease or other stress (yellowing or browning, withering, spots, etc.). Some of these symptoms may occur naturally as the plants mature, especially after seeds have matured. If the water level is satisfactory, obtain guidance from a local agricultural extension agent, or knowledgeable garden center.

- Manually pick large insects (e.g., caterpillars, slugs) causing damage to the wetland vegetation. For serious insect infestation, which is destroying the vegetation, a chemical agent may be applied after obtaining guidance from a knowledgeable person (e.g. agricultural extension agent, or good garden center) for proper chemical and application rate. If vegetation does not appear healthy and water levels are correctly maintained, add a balanced liquid fertilizer periodically (three times a growing season) to the wastewater by flushing down a toilet. “Normal” domestic sewage may not contain all the trace nutrients and elements required by the vegetation in a gravel substrate. Replace dead plants as necessary to fill voids. Pull up “volunteer” weeds, trees and shrubs from the wetlands. These species will shade and crowd the desirable wetland plants.

- Prevent excessive shading of wetland vegetation by controlling growth of trees or high shrubs near the wetland cells. Most wetland plants need at least six hours of sunshine each day. Remove mature wetland vegetation after the plants have browned in the fall if desired for visual aesthetics. However, only cut approximately two-thirds of the height of the plants. The removed material may be laid on the bed surface as mulch.

- Encourage deep root growth by lowering the water level over several weeks during the dormant vegetation period. Do not drop the water level too low, too quickly and leave the roots without water. After frost has killed the top of the plants, drop the water level below the gravel surface to one-third the gravel depth (e.g., 4 inches for a 12 inch depth) for a week; raise the level back to 1 inch below the surface for a week; drop the level two-thirds the depth (e.g., 8 inches for a 12 inch depth) for a week; again raise the level to 1 inch below the surface for a week; drop the level to 1 inch above the cell bottom (11 inches for a 12 inch depth) for a week; raise the level to 1 inch. Repeat this cycle once more.

- Divide and replant decorative flowering species (e.g., iris) to enhance the system attractiveness.

Odor Control (TVA)

- Standing water on the substrate surface is the probable cause of objectionable odor. Level any low and high spots on the substrate surface, which create small standing pools by raking and/or filling with additional substrate. If a too high water level is causing standing water on most of the substrate surface, lower the level using the water level control device so that it is about one inch below the substrate surface. Odors will also occur from water standing or flowing within the water level control structure and open observation standpipes. Odors from these structures should be noticeable only when the caps or covers are removed or loose. Secure the caps and lids in place to prevent these odors from escaping.

Drain Field (TVA)

- Mow to keep the area attractive.

- Fill in any low areas where surface water ponding occurs.

- If wastewater surfaces above the drainfield for extended periods, check risers to ensure that all extensions are receiving water. If any section is not receiving water, a pipe may be separated or crushed. Repair as necessary.

- Installation of water conserving plumbing fixtures or additional drain field area may be necessary.

Health and Safety (TVA)

- Prevent children from playing in the system to avoid contact with potentially infectious microorganisms.

- The tight fitting lid on the water level control structure may be secured with a latch and lock if a potential safety problem is a strong consideration.

Miscellaneous (TVA)

- Leaky Plumbing Fixtures - Repair faulty plumbing fixtures as soon as they are noticed. Leaky or stuck commode flaps can particularly reduce treatment effectiveness of a small CW due to the large quantity of water that can be lost in a short time period.

- Household Chemicals - Do not empty strong chemicals (e.g., some drain cleaners, floor cleaners, bleach) into the sanitary system. Chemicals can upset the septic tank causing excessive solids to wash out of the septic tank and possibly plug the substrate. Also, chemicals can damage and kill the vegetation.

- Pipe Clogging - Prevent or minimize pipe clogging by restricting flushing of grease, food particles, and tampons and other personal hygiene products. Use cleanouts installed before the septic tank and the wetland cells to unclog pipes.

- Herbicides/Pesticides - Do not apply herbicides and pesticides that can damage vegetation either on or near the system.

- Mulch - Maintain a three-inch mulch layer on top of the substrate, either with litter from the wetland vegetation, pine straw or bark, or other suitable material.

- Surface Drainage - Reroute any surface drainage entering the CW around or away from the cell (s).

- Animals - Prevent animals from digging in CW, destroying vegetation and making holes in the substrate and mulch. Unusual Problems - Contact your local county health department for guidance if any unusual problem is noted.

12.4.2 Observed Conditions

The site used for this study was well maintained and many of the recommendation in the previous section were followed. The grass was cut, berms maintained and in general the site looked very good and no odors were detected above ground. This site provided insight as to the changes in system maintenance and operation that differing ownership can produce. Approximately halfway into the sample period the residence was sold to another four-member household. On February 28, 1998, the previous owner vacated the residence, on March 6, 1998, the new owners moved in. System loading remained about the same, but with a lower daily average flow, 187 gallons per day versus 228, an 18 percent reduction. This reduction in flow created a situation during the summer months where there was a period of no flow through the wetland. The observed water level in the constructed wetland fell to 3 inches below its normal level as set by the standpipe in the effluent sump. This did not appear to have an adverse affect on the vegetation and within three months the water rose back to its normal level. Water levels fluctuated during this period depending on rainfall at the site. During the previous year at an identical time period this cessation of flow was not observed with the other occupants. It is believed that active system maintenance was not performed by either of the owners with the exception of topping off of the surface flow wetland polishing pond by the first owners as needed during hot weather. At no time during the 16-month sample period was the level of the surface flow wetland polishing pond observed to be high enough to overflow to the leach field. This indicates that with no known leaks in the system there was no impact on ground water. All waste water from this system was lost to evaporation or evapotranspiration.

The first owners (constructed the system) paid very close attention to the need for maintenance. Makeup water was provided during periods of low flow and when the family was out of town. This assured that the plants received enough water to survive especially during the hot dry season. The second owners paid almost no attention to the system in this respect. It is not known if operation information was given to the new owners. There is no formal mechanism for assuring that this is accomplished for real-estate transactions involving alternative systems.

12.5 Reported System Performance

All states in the US allow the use of constructed wetlands for onsite wastewater treatment applications. TVA developed the first design guidelines focused exclusively at small systems. Louisiana, Mississippi, and Arkansas have standard sizing and designs that can be applied to all areas in their respective states. Nationally, municipal constructed wetlands systems appear to have a poor record for nitrogen removal in general and ammonia treatment in particular (Askew, Hines and Reed, 1994). Kadlec and Knight (1996) summarized performance of all sizes of SSF constructed wetlands operating in North America. This data indicated mean pollutant removal efficiencies for BOD_5 (69 percent), TSS (79 percent), TN (56 percent), TKN (50 percent), NH_3-N (25) percent), and NO_3-N (69 percent). Effluent data indicated mean pollutant concentrations for BOD_5 (8.6 mg/L), TSS (10.3 mg/L), TN (8.41 mg/L), TKN (7.16 mg/L), NH_3-N (4.51 mg/L, and NO_3-N (1.35 mg/L). Burgan and Sievers (1994) reported BOD and TSS removals of 87 and 65 percent, respectively for onsite-scale SSF constructed wetland systems. Ammonia and phosphorus removals were determined to be 44 and 42 percent, respectively. Huang et al., (1994) reported that several test, onsite SSF constructed wetlands operated with a detention time of 3 to 5 days resulted in pollutant removals of BOD_5 (63 -71 percent), TKN (45-55 percent), NH_3-N (25-45) percent), and NO_3-N (54 percent). These investigators found no difference in performance based on differing plants or the use of simple pumped recirculation. Askew et al., (1994) reported that an SSF constructed wetland retrofitted with a vertical flow trickling filter and pumped recycle increased system performance especially for nitrogen removal.

Recent studies of various sized SSF constructed wetland systems operating in New Mexico (Thomson et al., 1996), indicated that many of these systems did not meet performance expectations for BOD_5, TSS, ammonia, nitrate and total nitrogen. Zachritz and Hanson (1998) have indicated that influent concentrations of pollutants (BOD, TSS, and ammonia) encountered in New Mexico SSF constructed wetland systems are significantly higher than system data reported for these values in the national database.

12.6 Field Trial Results

12.6.1 Flow Characterization

A detailed flow characterization study was conducted for this site from November 24 through December 17 (Julian day 328-351) 1998. This study provided information about the flow patterns from household activities based on hour to hour and day to day variations. The system was monitored with a Campbell Scientific CR500 data acquisition system (DAS) for a 3 week period where all flow events were recorded. During the 23 day test the DAS monitored 483 discrete pump cycles. Some of these pump cycles were the result of an accumulation of more than one small flow event. Other pump cycles were part of large flow events that would occur during bathing and laundry activities. The overall frequency of flow events as indicated by the number of pump cycles is shown in Figure 12.2. The most notable aspect of this chart is the degree of uniformity of pump cycles relative to the time of day. The large volume of the wetland has a tendency to dampen rapid pump cycling as the water slowly moves through the system. However, incidents of large usage are still readily discernible early in the morning and later in

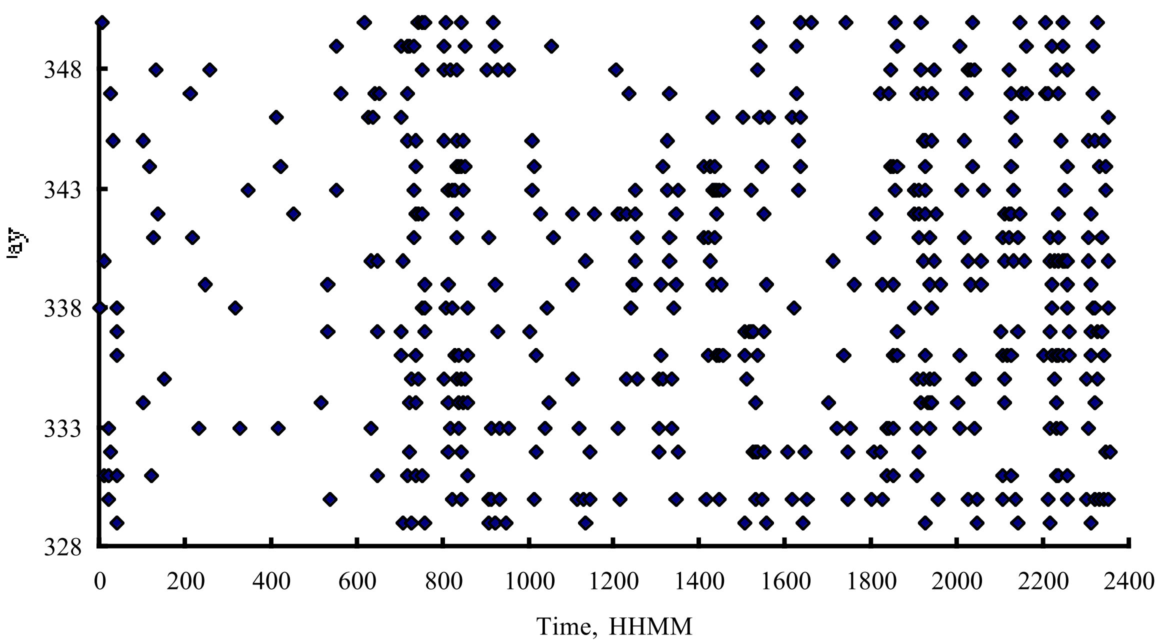

Figure 12.2. Frequency of Pump Events Recorded During the Flow Characterization Study.

the evening with scattered large events occurring on some days. The mean hydraulic hourly load profile for this site is illustrated in Figure 12.3. Hourly usage rates at this residence varied less than those of other monitored systems. Part of this consistency may be due to the nature of the household, a four-person household with one wage earner and one homemaker plus two school age children.

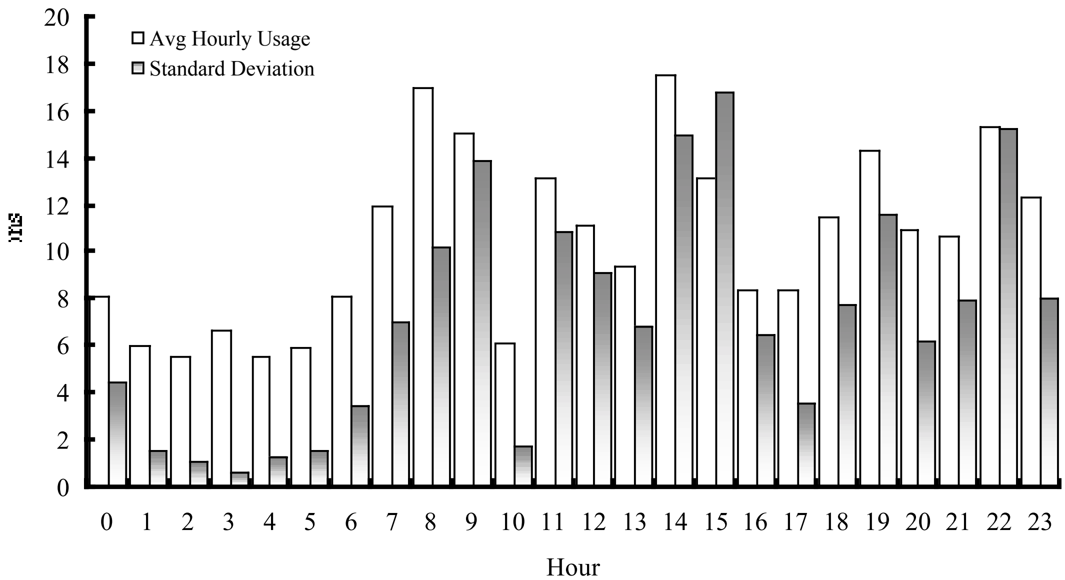

Figure 12.3. Mean and Standard Deviation of Daily Flows Recorded During the Flow Characterization Study.

In addition, this data indicates that our sampling program which used a composite of flows collected from 4:00 PM to about 9:00 AM captured about 69 percent of the flow based on time. Sampling did not collect a number of events that occurred during the mid-part of the day. The time required to accomplish this collection could not be incorporated in the study.

Mean flows determined for days of the week during the characterization study are shown in Figure 12.4. This data indicates that the mean flows varied from very small amounts (56 gallons) to large amounts (309 gallons). No apparent or re-occuring pattern of usage from day to day was observed in this household. The mean daily flow recorded for this period was 150.1 gallons per day.

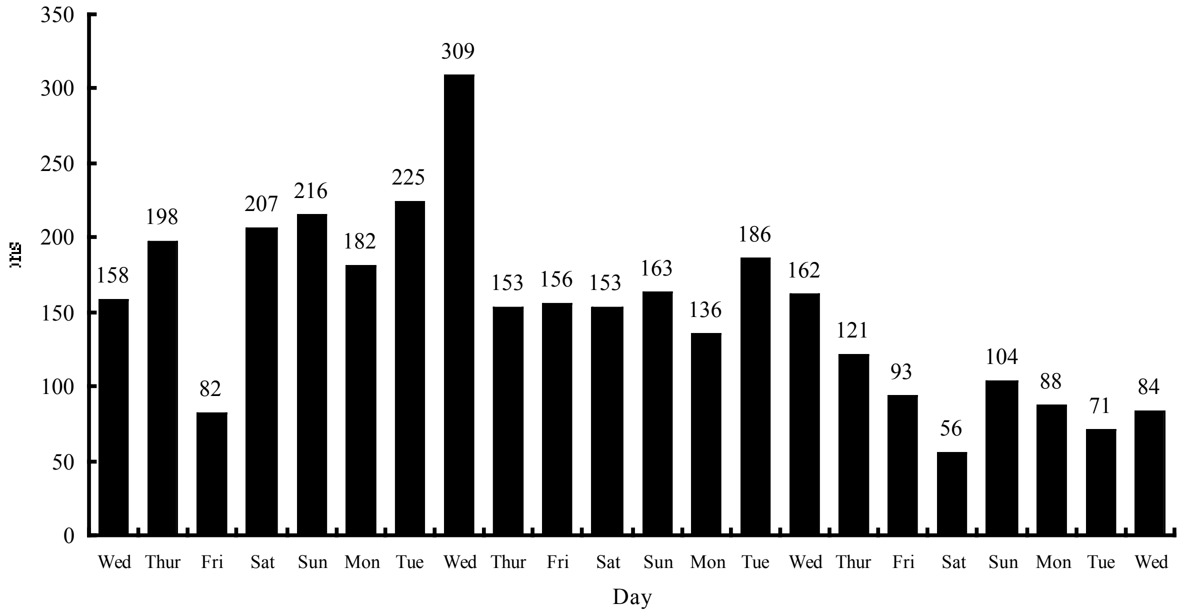

Figure 12.4. Mean Daily Flows for the Flow Characterization Study.

12.6.2 Hydraulic Analysis

As mentioned in the previous Chapter, the hydraulics of most reactor vessels fall somewhere between complete mixed flow (CMF) and plug flow (PF), which are the two ideal reactor models. This is termed non-ideal flow, which can be described by two basic models, the plug flow with dispersion model and the tanks in series model. The flow pattern for the wetlands system was analyzed by determining the residence time distribution (RTD) of material flowing through the vessel. The method for the hydraulic analysis for the wetlands system is given in the Methods (Chapter 8) section of this report.

The tracer study for this system was conducted from November 24 to December 17, 1998. On November 24, 1998, a total of 1,134 grams of bromide (Br) were introduced into the wetlands system as a sodium bromide solution by the methods outlined in Chapter 8. During the tracer study, the concentration (C) of Br on the system effluent was measured and recorded as a function of elapsed time in hours, from the beginning of the input of tracer into the system. This concentration vs. time (C vs. t) data was used directly and in conjunction with flow models to predict actual system behavior in terms of detention time and flow patterns. The C vs. t data for the wetlands system was used to construct a C vs. t curve, which is shown in Figure 12.5.

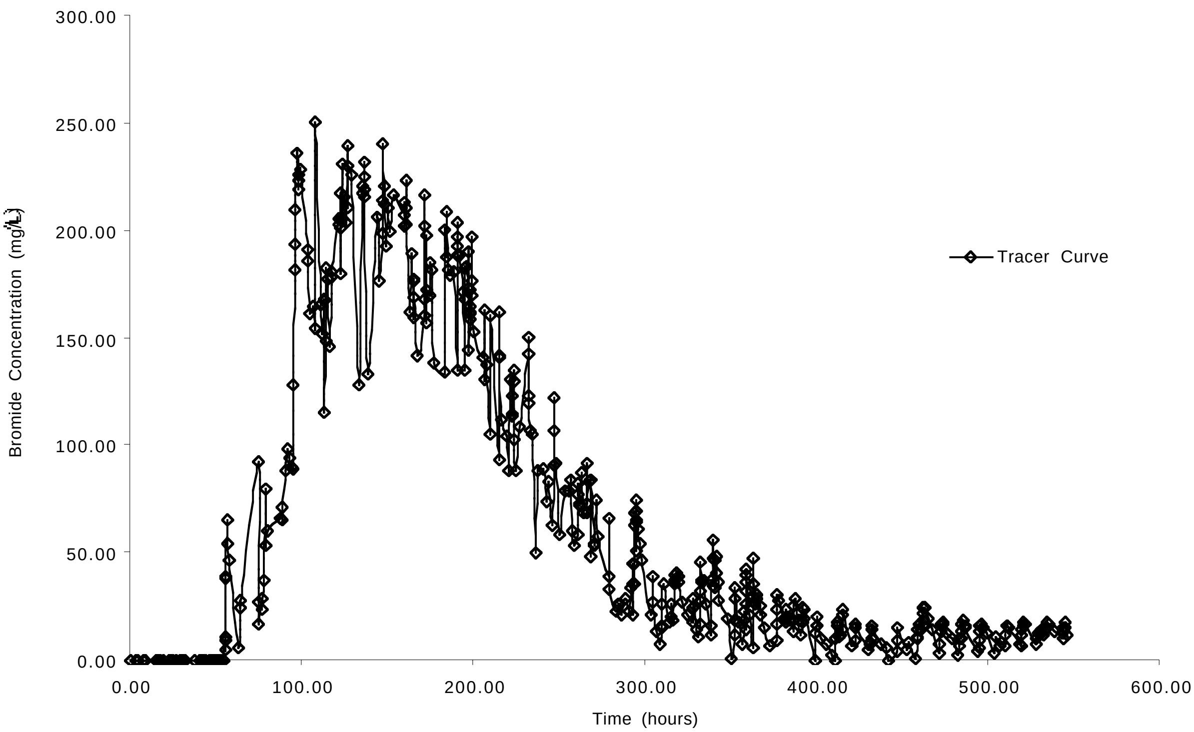

Figure 12.5. Wetlands System Tracer Study Concentration vs. Time Curve.

In addition, the overall shape of the C vs. t curve was used to look for evidence of short-circuiting in the system. Based on the shape of the C vs. t curve for the wetlands system, there was no evidence of short-circuiting, which would have shown up as a sharp and early peak.

As mentioned in the previous Chapter, the mean and variance of a tracer curve are directly related to the system detention time and are the two quantities for describing tracer curves that are used in all areas of tracer experimentation. As previously mentioned in Chapter 8, the mean is the mean detention time of the system, while the variance tells how spread out in time the curve is. Using equation 8.3 (Chapter 8) the calculated mean detention time for the artificial wetlands system is as follows:

$$\text{Mean} = \sum(t_{i+1} + t_i)\,(C_{i+1} + C_I)\,(t_{i+1} - t_i) / 2\sum (C_{i+1} + C_i)\,(t_{i+1} - t_i)$$
$$= 192.47 \text{ Hours}$$

The variance was calculated using equation 8.4 and is as follows:

Variance = $(\sum (t_i + t_{i+1})^2 (C_i + C_{i+1}) (t_{i+1} - t_i) / 4\sum (C_i + C_{i+1}) (t_{i+1} - t_i)) - \text{mean}^2$

$= 8{,}830.80 \text{ hours}^2$

The calculated mean, 192.47 hours, along with the C vs. t data was used to construct the E curve for the system using the methods outlined by Levenspiel (1993) and which were discussed in general in Chapter 8. Namely, $E_t = C_i/\text{area}$, and $E_\theta = E_t * (\text{mean})$ and $\theta = t/\text{mean}$. The E curve for the wetlands system is shown in Figure 12.6.

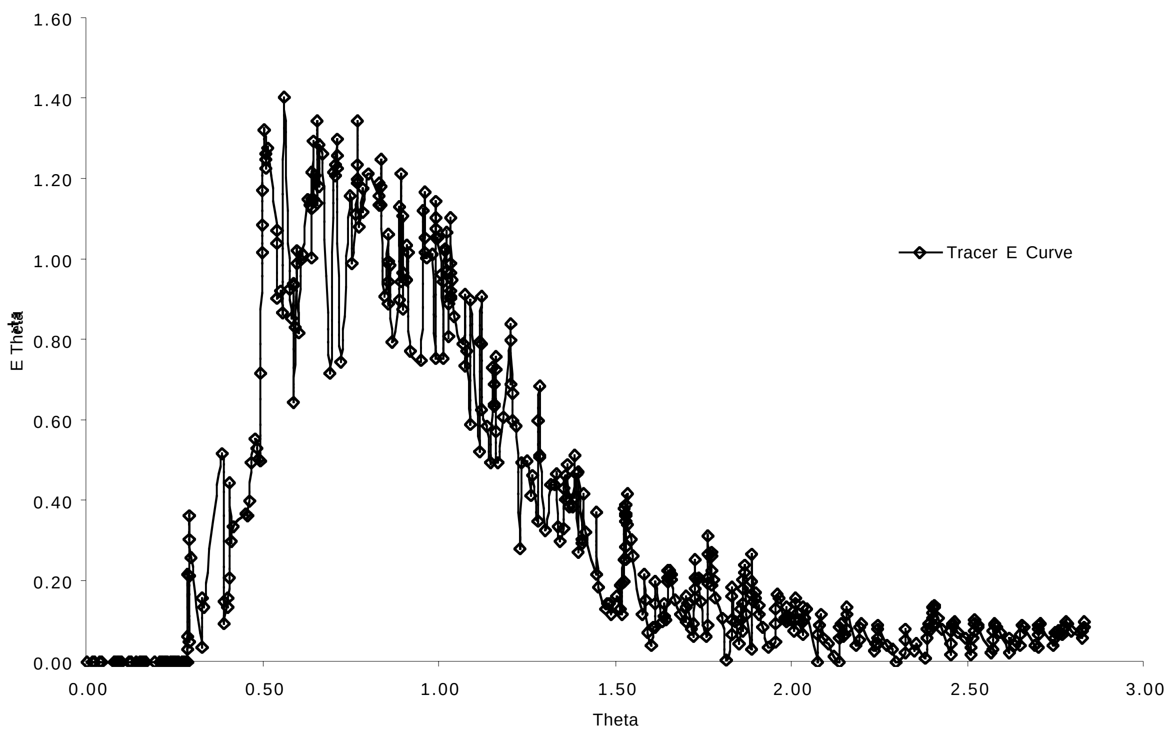

Figure 12.6. SSF Wetlands System E (θ) Curve.

To determine which flow pattern approximated the wetlands system, the E curve generated from measured C vs. t data was compared to theoretical E curves, such as the ones shown in Figure 11.8 of the previous Chapter. From this comparison, it was determined that the wetlands system E curve did not approximate the ideal complete mix flow region or the ideal plug flow region. Therefore, the artificial wetlands E curve fell somewhere between the two extremes of complete mix flow and plug flow, namely, the intermediate region that can be modeled by plug flow with dispersion or complete mix reactors in series.

As mentioned in the previous Chapter, theoretical E curves (Levenspiel, 1993) like the ones shown on Figure 11.8, were developed for different vessel dispersion numbers (D/uL values). These theoretical E curves were constructed for the closed vessel situation by numerical methods using equation 11.2 in the previous Chapter. For the constructed wetlands system the assumption of closed vessel was also made since the tracer entered and left the system in small pipes relative to the total volume of the system.

The next step in the wetlands system hydraulic analysis was to simulate the system using the Impulse program to determine how well the actual system approximated the plug flow with dispersion model. The Impulse program was used to simulate the wetlands system as a plug flow reactor with dispersion. The Impulse program utilizes equation 11.2 to determine the theoretical best fit curve to the experimental C vs. t data by running the program in the regression mode with regressable parameters or variables, such as inlet flowrates, reactor volume, dispersion number and inlet concentrations. The fit of the experimental data is based on equating the variances of the two curves about the center of gravity (mean residence time) of the distribution (Weber, 1972). The best fit of the simulated curve vs. the experimental or measured (tracer) curve was determined by visual inspection from the plotted Impulse output of C vs. t data.

Only one scenario and one model (PF with dispersion) were used to simulate the artificial wetlands system. The reason for using only one model was because upon visual inspection and comparison to the theoretical E curves in Figure 11.8 the E curve for the artificial wetlands system approximates the plug flow (PF) with dispersion extreme rather than the complete mix flow (CMF) extreme. The scenario where the volume was varied was the only one used for this system, simply because this scenario closely resembled actual conditions based on percent bromide tracer recovery and flowrate into the system. Also, the reason for using two scenarios for the Whitewater system tracer study was mainly to see which scenario gave the best fit curve and also matched the actual system conditions. The match between actual system conditions such as inlet flowrate, volume, and percent recovery of bromide tracer is more important than the criteria of best-fit curve to the experimental data. Therefore, in the Whitewater system tracer study, it was determined that the results obtained with the scenario where volume was varied were representative of the system. Furthermore, for the Whitewater system the reason for the worse curve fit when compared to the scenario where flowrate was varied was attributed to an error in experimental data caused by material deposition on the bromide probe.

During the artificial wetlands tracer study, the error caused by material deposition on the bromide probe was corrected by periodic monitoring of the probe calibration curve and thus the drift from the start of the experiment was determined. Based on the observed drift in the calibration curve, a correction factor was determined and applied to the C vs. t data. The corrected C vs. t data resulted in an excellent curve fit with the Impulse generated C vs. t data, as well as good correlation of the actual system conditions of inlet flowrate, percent recovery of bromide tracer and volume. This correction eliminated the need for a second scenario to be used in the Impulse simulations. The scenario used to simulate the artificial wetlands system, consisted of holding the inlet flowrate and concentration constant with the flowrate at 25.8 liters/hour (163.6 gallons/day), which was the actual calculated average flowrate and the concentration necessary to insure a pulse input of 1,134 grams of bromide tracer into the system. The reactor vessel volume was varied by Impulse.

Figure 12.7 shows a plot of the simulated curve vs. the tracer curve constructed by using the output of C vs. t data from Impulse. By visual inspection, the curve fit is excellent, the Impulse calculated dispersion number (D/uL) for this scenario was 0.06, versus the calculated value of 0.138 using the actual C vs. t tracer data. The actual average influent flow-rate during the tracer study was calculated to be 25.8 liters/hour (163.6 gallons/day). This value was calculated based on the total pumped volume divided by the total time. The reactor vessel volume calculated by Impulse was 4,187 liters (1,106 gallons), which should be comparable to the wetlands system void volume or sometimes referred to as the water volume. The actual system water volume was not calculated and is dependent on the size of rock used as fill. Furthermore, all wetlands possess a depth distribution, due to unavoidable irregularities in the as-built condition, which in turn is due to the microtopography of a natural wetland (Kadlec and Knight, 1996). Therefore, it is rarely possible to perform a quantitative fill or drain experiment to quantify the water volume. However, the total system volume is approximately 5,140 gallons, which would mean that if the Impulse calculated water volume of 1,106 gallons is fairly

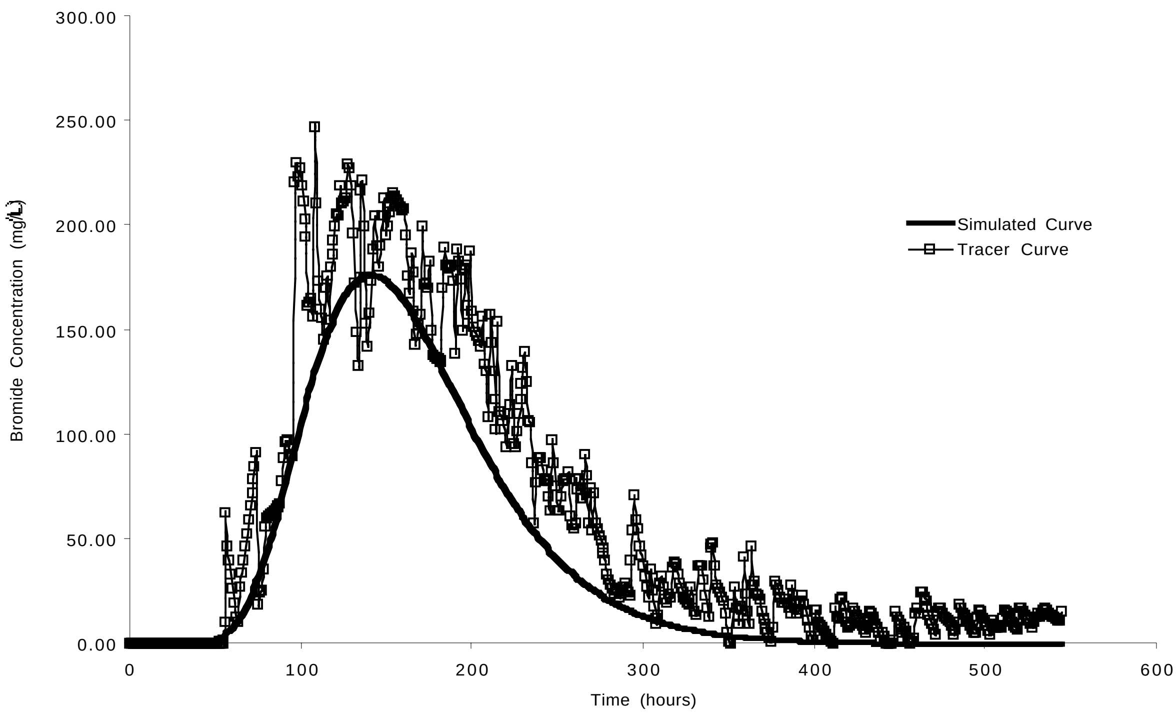

Figure 12.7. Tracer Curve vs. Plug Flow w/Dispersion Simulated Curve for the Wetlands System - Calculated D/uL = 0.06.

accurate, the system has a void fraction of about 21 percent, which is below the values of 28 to 37 percent used for design. The investigator's experience with SSF system indicates that some loss of pore space can be expected as the bed matures.

Based on the results of the Impulse simulation of the artificial wetlands system as a plug flow with dispersion reactor, the dispersion numbers (D/uL) of 0.138 (C vs. t measured data) and the value of 0.06 calculated by Impulse show that the deviation from plug flow is also large (D/uL>0.01). However, the wetlands system is closer to the plug flow extreme than the Whitewater system previously discussed. Furthermore, the shape of the tracer curve is consistent with other tracer studies done on wetlands systems, for example the tracer curve of a submerged flow (SF) wetland EW3 at Des Plaines, IL in August 1991, which is shown in Kadlec and Knight, 1996, clearly resembles Figure 12.5. However, this particular tracer curve was simulated using four complete mixed flow tanks in a series model, rather than a plug flow with dispersion model. As was mentioned in the previous Chapter, the plug flow with dispersion model and the complete mixed flow reactors in series models are roughly equivalent.

12.6.3 Water Quality Analysis

The SSF constructed wetlands system was performance tested for 65 weeks from September 2, 1997 through January 7, 1999 with 32 influent and 27 effluent samples collected. This system involved sampling the effluent from the household septic tank and the effluent from the first stage SSF constructed wetlands. Samples were not taken from the raw influent and samples were not taken from the second stage polishing pond. The average daily influent flow shown in Figure 12.8, indicated that flow varied between 135 and 328 gpd over this period with an overall average flow of 201.1 gpd. The effluent varied between 0 and 397 gpd over this period with an overall average flow of 139.9 gpd. Influent flows appeared consistent over the study period. The effluent flow was impacted by evapotranspiration (ET) losses from plants and illustrated marked seasonal variation. Flows in the winter months were nearly the same as influent flows, but as evaporation increased in the warmer months (5/1/98 to 12/1/98) the effluent flow decreased to a point of zero discharge from the system. Thus, ET appeared as an active factor in system performance for six months of the year. ET can affect removal kinetics and result in the concentration of pollutants. The summary of the flow data and the impact on the system is provided in Table 12.1.

Table 12.1 Summary of Different Flow Conditions and Unit Operation Detention Times for the Test System.

Parameter	Design Flow	Mean Flow	Flow Study	Tracer Study
Flow, gpd	350	201.1	150.1	163.6
[1]**Total Unit (liquid) Volume, gal**	2,145	2,145	2,145	2,300
System Detention Time, days	6.1	5.61	6.76	6.14
% Difference from Design	0	67.8	73.2	70.6
Reactor Volume, gallons	945			
Reactor Detention Time, days	2.7	4.45	5.36	4.88
Clarifier Volume, gallons	1,200	1,200	1,200	1,200
Clarifier Detention Time, days	3.4	5.96	7.99	7.33
Clarifier Overflow Rate, gpd/ft^2	8.72	5.01	3.74	4.08

[1] Assumes wetland cell had 2.5 day detention time with 350 gpd.

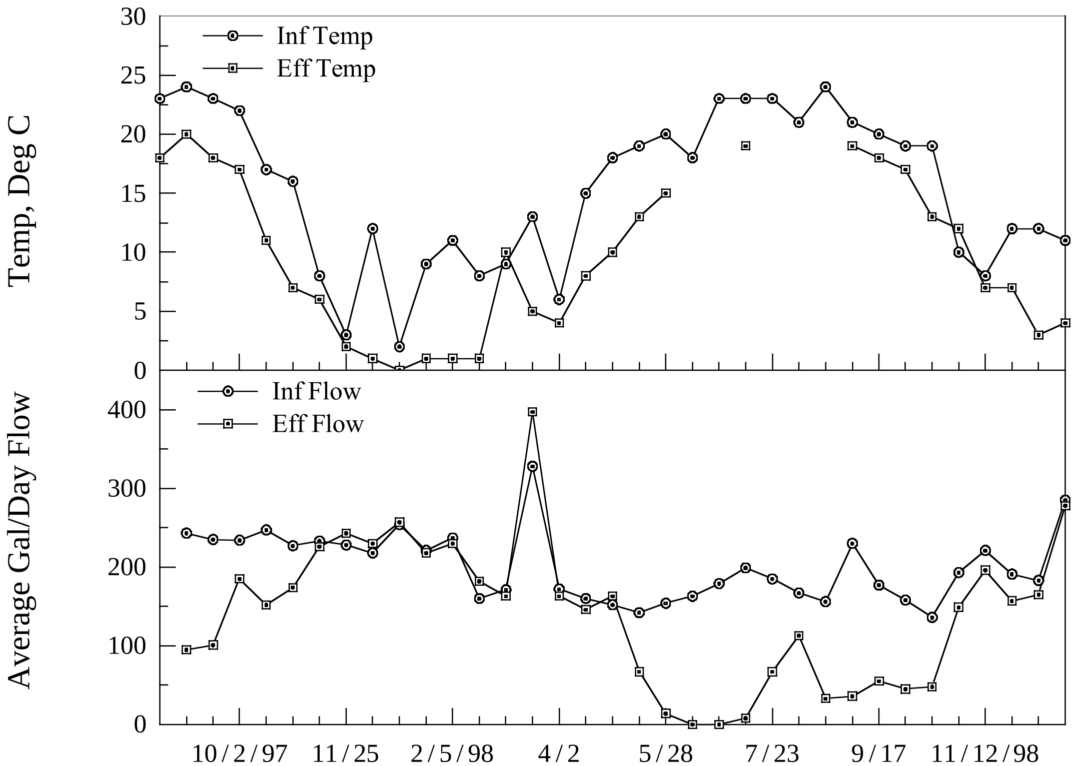

Figure 12.8. Subsurface Flow Constructed Wetland System Experimental Data.

Influent and effluent temperature varied from 2 to 24 °C and 0 to 20 °C, respectively over the study period. Mean temperature data (Figure 12.8) for the site indicated influent and effluent values of 15.5 and 9.6 °C, respectively. Both sample locations were more exposed to the elements than the Whitewater site. In addition, the SSF constructed wetlands have a large surface area exposed to the elements and could be expected to experience a greater heat loss and drop in operational temperature. Some design guidelines suggest that in cold areas, surface mulch be used on the wetlands to minimize heat loss. These low temperatures can affect the performance of biological treatment process particularly nitrification. Kemp and George (1996) suggests that nitrification will cease at temperatures below 12 °C.

The electrical conductivity (EC) reflects the total dissolved solids in a particular water sample. As previously mentioned, the change in EC can be an indicator of evaporative processes or the addition of chemicals such as from a water softener, laundry operations, reverse osmosis unit, electroplating, or photo developing processes. Many of these processes may add chemicals that cannot be detected by other measurement techniques or require very specialized and expensive analysis. The EC for influent and effluent samples for the SSF constructed wetlands test site are shown in Figure 12.9. The influent samples varied from below 200 to over 1,000 with lower values occurring in the winter months and higher values occurring in the summer months. This increase is difficult to explain since no increases were noted in sulfates or chlorides during those periods. The effluent data indicated that during the winter months when ET was low, influent and effluent EC was nearly equal. However from mid April through the beginning of December, effluent EC was higher than influent EC. During a part of the summer months a zero discharge condition was experienced and no water quality data was recorded for that period. When flow resumed EC values were elevated and slowly decreased until early winter. Because of the ET affect during the warmer months some concentration of pollutants will occur in SSF constructed wetlands. The mean EC for influent and effluent was 630.1 and 709.4 S/m, respectively. There was no significant difference in these measured values (p-value = 0.3079). Chloride data (Figure 12.9) for the influent and effluent averaged 90.9 and 135.0 mg/L, respectively. Influent concentrations did not vary over the study period. Effluent chloride appeared to be concentrated through the wetlands cell especially during the high ET periods as noted for the EC data. Chloride is not taken up by plant processes and thus is a conservative tracer. Sulfate data exhibited some variability in the influent. As previously mentioned sulfates are contained in sulfuric acid (can be used for scale removal) and drain cleaners. Sulfates are also not utilized by plant processes. Sulfate can be converted to hydrogen sulfide under highly reduced or anaerobic conditions and can be removed via this process. Sulfate showed a similar pattern in the effluent to Chloride and EC. The influent and effluent sulfate concentrations averaged 23.2 and 39.2 mg/L, respectively and were not significantly different (p-value = 0.11430).

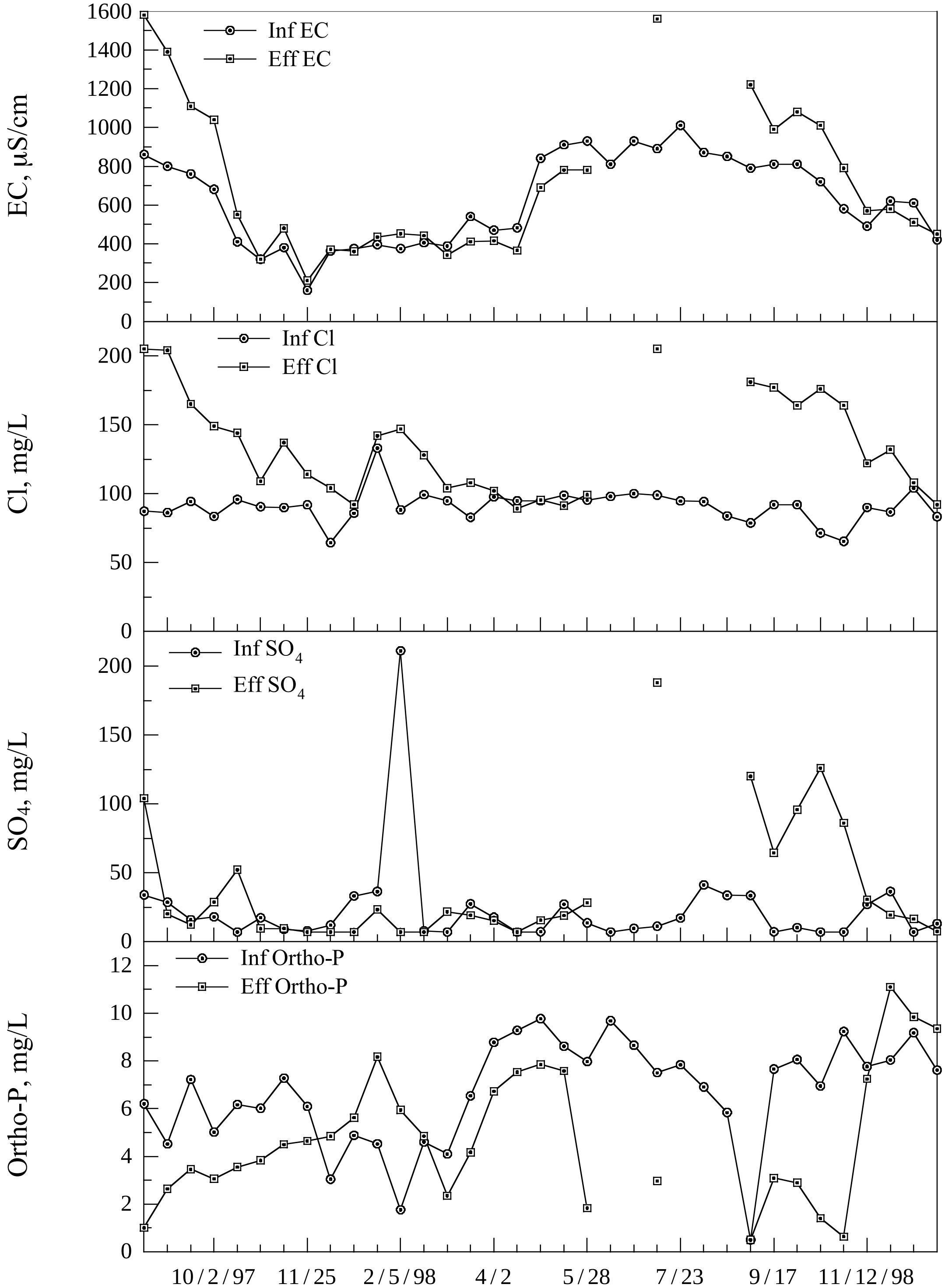

Figure 12.9. Subsurface Flow Constructed Wetland System Experimental Data.

Chloride data (Figure 12.9) for the influent and effluent averaged 90.9 and 135.0 mg/L, respectively. Influent concentrations did not vary over the study period. Effluent chloride appeared to be concentrated through the wetlands cell especially during the high ET periods as noted for the EC data. Chloride is not taken up by plant processes and thus, is a conservative tracer. Sulfate data exhibited some variability in the influent. As previously mentioned sulfates are contained in sulfuric acid (can be used for scale removal) and drain cleaners. Sulfates are also not utilized by plant processes. Sulfate can be converted to hydrogen sulfide under highly reduced or anaerobic conditions and can be removed via this process. Sulfate showed a similar pattern in the effluent to Chloride and EC. The influent and effluent sulfate concentrations averaged 23.2 and 39.2 mg/L, respectively and were not significantly different (p-value = 0.11430).

Dissolved or ortho-phosphorus concentrations in the influent and effluent from this test system (Figure 12.9) averaged 6.7 and 4.8 mg/L, respectively. The influent and effluent concentrations were significantly different ($p = 0.001039$) with a system percent removal of ortho-phosphorus of 38.9 percent. This is similar to the phosphorus removal value of 42 percent for SSF constructed wetlands reported by Burgan and Sievers (1994).

The data for pH is shown in Figure 12.10. Maintaining near neutral pH (6 to 8) is important for the stability of biological processes. Some biological processes such as nitrification can reduce pH. Influent pH values ranged from 7.2 to 8.3 over the course of the study while effluent values ranged from 7.1 to 8.5. The mean influent and effluent pH values were 7.8 and 7.5, respectively and were not significantly different.

The TSS data values for the system shown in Figure 12.10 indicated consistent, but variable influent concentrations with a single extreme value of 400 mg/L noted. The average effluent concentration was determined to be 40.83 mg/L with a standard deviation of 62.1. The wastewater measured was septic tank influent and thus much of the variation was eliminated by the upstream septic tank. The effluent values averaged 55.47 mg/L with a standard deviation of 157.89. The influent and effluent were not significantly different at a p value = 0.615082. Two extreme effluent values of 400 and 800 mg/L were experienced over the study period. If these extreme values for influent and effluent are eliminated from the data set as outliers, the resulting influent and effluent mean values are 30.6 and 16.7 mg/L, respectively. Under these conditions, the influent and effluent were significantly different at a p value = 0.002264 and the calculated

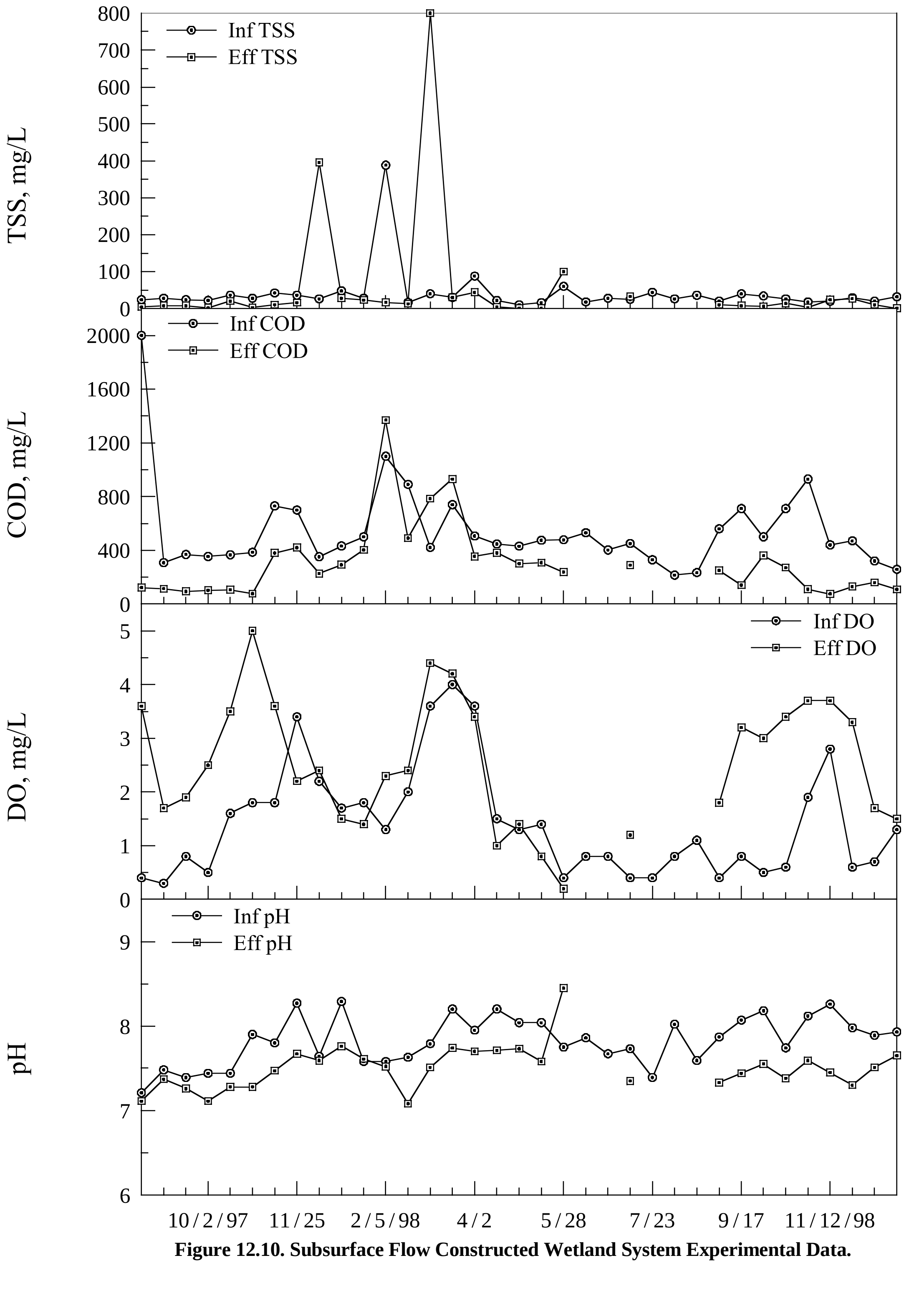

Figure 12.10. Subsurface Flow Constructed Wetland System Experimental Data.

percent removal TSS was 45 percent. This is lower than the value of 79 percent removal reported by Kadlec and Knight (1996). This also indicated that TSS was consistently below 20 mg/L, but did not attain a 10mg/L standard or less**.**

The BOD_5 values for this system were not graphed in part because only half the values were provided by the laboratory after samples had been submitted. Many of these were reported as over range or under range and were not usable. The BOD_5 values for the influent ranged from 116 to 770 with a mean of 207.6 mg/L. The BOD_5 values for the effluent ranged from 6 to 282 with a mean of 96.0 mg/L. The influent and effluent were significantly different at a p value = 0.000128. The percent removal was 53.7 percent over the study period. This is lower than 87 percent removal of BOD reported by Burgan and Sievers (1994).

The COD data values for the system are shown in Figure 12.10. The average influent concentrations were determined to be 543.9 mg/L with a standard deviation of 323.6. The effluent values averaged 313.0 mg/L with a standard deviation of 281.5. The influent and effluent were significantly different at a p value = 0.00340. The calculated percent removal of COD was 42 percent for this system. Calculated BOD to COD ratios can be useful to evaluate the biodegradability of a wastewater. The BOD/COD ratio for the influent varied from 0.092 to 1.52, with an average value of 0.46. The BOD/COD ratio for the effluent varied from 0.049 to 0.826 with an average value of 0.35. These values are similar to values of 0.4 to 0.8 reported for domestic wastewater by Metcalf and Eddy Inc. (1991) and Laak (1986).

The DO data values for the system shown in Figure 12.10 indicated some variation in influent DO with concentration ranging from 0.3 mg/L to over 4.0 mg/L The average influent concentrations were determined to be 1.41 mg/L with a standard deviation of 1.03. The effluent values averaged 2.53 mg/L with a standard deviation of 1.19. The influent and effluent were significantly different at a p value = 0.000126. DO increased through the system.

A critical factor for the successful conversion of ammonia to nitrate is the availability of DO. It would appear that for this system DO was not a limiting factor for nitrification. Ammonia data for the influent and effluent is shown in Figure 12.11. The average influent concentrations were determined to be 70.1 mg/L with a standard deviation of 30.0. The effluent values averaged 39.6 mg/L with a standard deviation of 16.7. The influent and effluent were significantly different at a p value = 0.0000036. The calculated percent removal of ammonia was 43.5 percent for this system. This is very close to 44 percent removal of ammonia reported by Burgan and

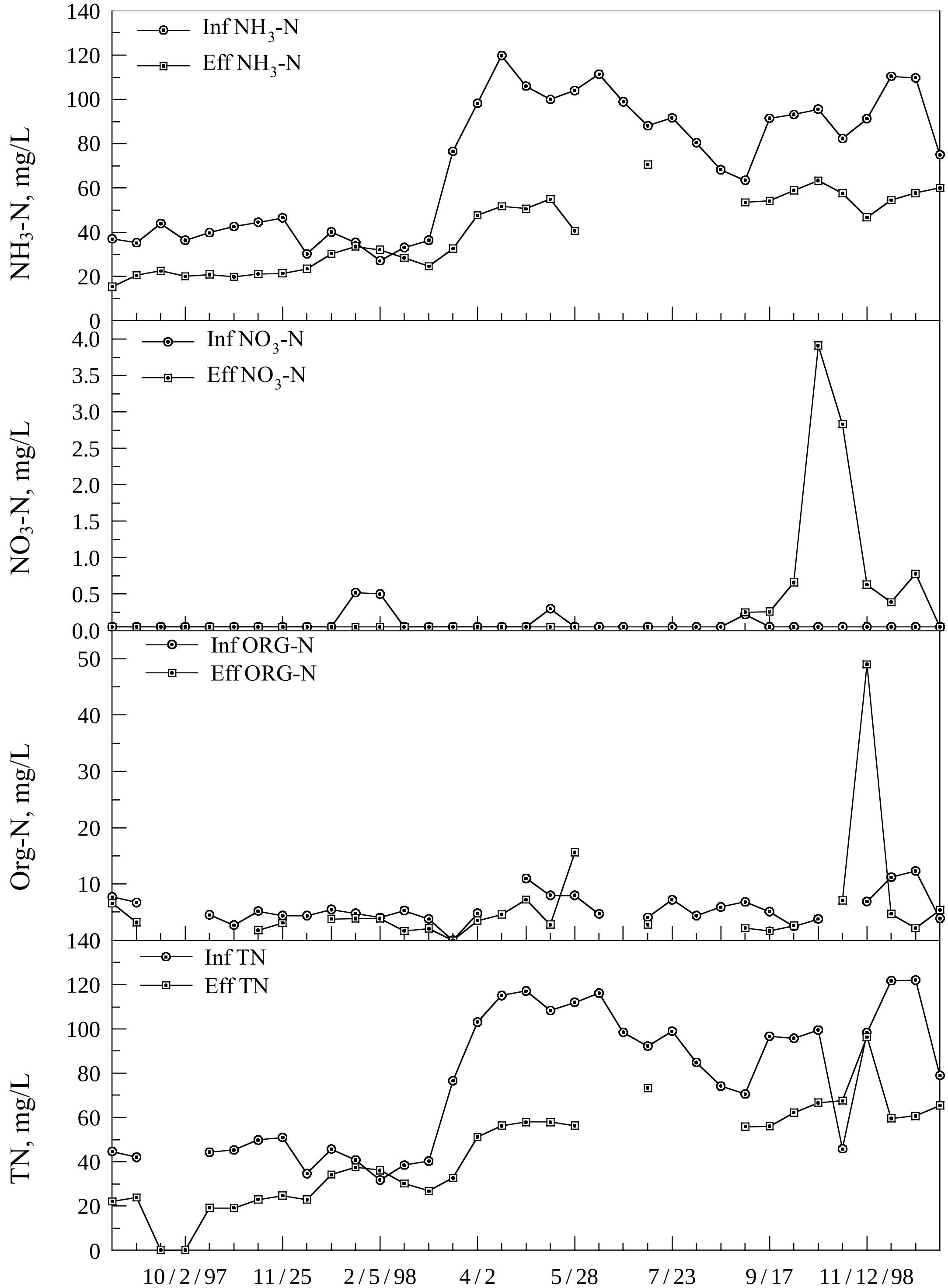

Figure 12.11. Subsurface Flow Constructed Wetland System Experimental Data.

Sievers (1994) and higher than the value of 25 percent removal reported by Kadlec and Knight (1996). Ammonia did increase with the different occupant in the dwelling. After about the middle of May 1998, a new family occupied the house and ammonia concentration increased from about 40 mg/L to slightly above 55 mg/L. Whether this was due to eating habits or other factors is not known.

Nitrate is normally not found in septic effluent or raw sewage because of the limited nitrification rates. Nitrate is the product of nitrification or can be added by chemical such nitric acid. Drinking water standards for nitrate are 10 mg/L as N. Nitrate data ($NO_3 + NO_2$) for the influent and effluent is shown in Figure 12.11. The average influent concentrations were 0.09 mg/L with a standard deviation of 0.12. The effluent values averaged 0.36 mg/L with a standard deviation of 0.85. Considering that it appears that the system does convert ammonia to nitrate, it does also appear that the process of denitrification was active in the system. The influent and effluent nitrate data were not significantly different at a p value = 0.0666772.

Organic nitrogen for the influent and effluent is shown in Figure 12.11. The average influent concentrations were 5.7 mg/L with a standard deviation of 2.62. The effluent values averaged 5.9 mg/L with a standard deviation of 9.67. The influent and effluent were not significantly different at a p value = 0.89715.

Total nitrogen for the influent and effluent is shown in Figure 12.11. The average influent concentrations were 76.8 mg/L with a standard deviation of 30.7. The effluent values averaged 43.2 mg/L with a standard deviation of 22.8. The influent and effluent were significantly different at a p value = 0.00000756. The calculated percent removal of ammonia was 43.8 percent for this system. This is lower than the value of 56 percent removal reported by Kadlec and Knight (1996). A similar increase was reported for TN when the house was old and occupied by a new resident. This single wetland cell system could not meet a TN standard of 10 or 20 mg/L if there was discharge to the subsurface. However it should be noted that the system never discharged to the subsurface. The second stage polishing pond successfully collected and evaporated all water that came from the dwelling. No discharge to the subsurface disposal was recorded over the study period. As designed the system is a zero discharge facility.

The fecal coliform (FC) data for the wetlands system was highly variable with mean influent and effluent values of 3.43×10^5 and 1.48×10^4 cfu/100mL, respectively. This resulted in a removal rate of over 95.6 percent. Influent values ranged from 4.20×10^3 to 5.30×10^6 and effluent ranged from 350 to 1.54×10^5cfu/100mL.

12.7 Conclusions

The SSF constructed wetlands system was performance tested for 65 weeks from the September 2, 1997 through January 7, 1999 with 32 influent and effluent samples collected. The flow characterization and reactor tracer analysis was also performed on the system. In addition, the installation, maintenance, and operation of the system were evaluated. The system as tested evaluated performance based on effluent from the septic as the influent to the system. All other systems used raw wastewater as the influent. This would result in lower than normal percent removals because of the lower influent concentrations. Access to the raw wastewater in this established system was limited.

The hydraulic analysis of the system indicated no short circuiting or unusual flow problems with the system. The system exhibited a flow pattern that strongly resembled a plug flow with dispersion reactor. The average measured flow over the study period was 201.1 gpd, which was slightly higher than the flows measured during the tracer studies and flow characterization study. These flows were less than the estimated design flow (450 gpd) for the unit.

A summary of operating parameters for this system was shown in Table 12.1. A summary of performance data based on those operating conditions is provided in Table 12.2. This data indicates that the system as tested was operating below the estimated design flow and performance should be optimum especially for nitrogen removal. Normally if a passive wetlands system is designed for nitrogen removal the detention time of the system is increased from 2.5 to over 5 days. The detention time determined from the tracer study indicated that the system was operating at about an 8-day detention time during the period of the tracer study.

Data from Table 12.2 indicated that the loading of BOD and ammonia for this system was slightly higher than the national average of 26.0 and 6.3 lbs/ac/day, respectively for SSF systems (Kadlec and Knight 1996). TSS loading was very low compared to the national data base value of 42.9 lbs/ac/day and the TN loading was about the same (10.2 versus 11.8 for national database).

Table 12.2 Summary of Operating and Design System Parameters for SSF Constructed Wetlands System.

Parameter	Loading lbs/ac/day	Loading lbs/day	Effluent mg/L	Percent Removal %
BOD_5	29.0	0.34	96.0	53.7
COD	77.7	0.91	310.0	42.0
TSS	4.3	0.05	16.7	45.0
NH_3-N	10.2	0.12	39.6	43.5
TN	11.1	0.13	43.2	43.8
Ortho-P	0.8	0.01	4.8	38.9

The critical regulatory parameters for BOD_5, TSS, TN, and FC are shown in Table 12.3 for the test system. These data indicated that some of the average values generated for the system were well above the recommended performance standards for any of the zones listed (BOD and FC). TN was slightly higher than the upper range, but TSS was within the upper range suggested. The best data for the field trials indicated that all parameters except FC could meet the upper standard. TN was below 20 mg/L, but did not reach a 10mg/L standard.

Operationally, the SSF constructed wetland system, experienced none of the problems encountered with the aerated systems and no odors or ponding or plugging was noted in the system. A distinct advantage of this system is the lack of mechanical devices to fail and the continuous operation of the system. The testing did not reflect the overall performance either in terms of ultimate discharge or in terms of overall system percent removal. Ultimate discharge came from the polishing pond and would have been lower than the measured values from the first stage. Additionally, at times no flow was encountered from the first stage and this would have been a zero discharge system. No flow was ever observed from the second stage of this system and therefore, overall this wetland was a zero discharge system. No other system in the study came close to meeting this degree of treatment.

Table 12.3 Comparison of SSF Constructed Wetlands Data and Proposed Performance Standards.

	Field Trial Data		Performance Standards		
Parameter	Overall Mean	Best Results[1]	Zone A	Zone B	Zone C
BOD_5, mg/L	96.0	23.6	30	20	15
TSS, mg/L	16.7	2.0	30	20	15
TN, mg/L	43.2	14.2	30	20	10
FC, cfu/100mL	$1.48\text{x}10^4$	$1.31\text{x}10^3$	$1.0\text{X}10^4$	$1.0\text{X}10^3$	100
COD, mg/L	310.0	97.8	N/A	N/A	N/A
Ortho-P, mg/L	4.8	1.5	N/A	N/A	N/A

[1] Mean of lowest five data points (throwing out the lowest value) with recorded flow event

Evaluating this single cell of the wetland is complicated because the unit was not designed to stand alone. Newer systems are being designed to treat TN more effectively and these designs should not be discounted as not being an acceptable alternative treatment process. The cell tested in this study was an older design based mostly on BOD standards. It is clear that the system did operate at a lower flow scenario and based on detention time did approach some designs for wetlands that have been deployed with the notion of providing nitrogen treatment. The best data for the wetland cell as tested did meet the standard for TN of 20 mg/L or less, but did not meet the lowest standard for TN of 10mg/L or less.

Chapter 13 - Clearstream System

13.1 Site Description

The system tested was a Clearstream Model 500N Aerobic Treatment Unit. This system is characterized as suspended growth biological (SGB) treatment system designed to treat 500 gallons per day of domestic wastewater. A local manufacturer representative provided the unit and the design. The unit, which is less than 3 years old, was installed by an approved local onsite system installer. The test site was a three-bedroom house located in Bernalillo County east of the Sandia Mountains in a fractured bedrock region. The house was empty much of the time, which seriously compromised the data set. The home has a water softener for a portion of the water flow, and a garbage grinder is installed. The water softener backwash at this residence does flow to the onsite system. The leachfield system for effluent disposal appears to be a conventional system. The exact size and location of the leachfield are unknown. The system layout with sample sumps and required setbacks is shown in Figure 13.1. Sampling was set up using methods outlined in the previous methods section.

13.2 System Description

A typical Clearstream system consists of pretreatment tank (trash tank/septic tank), an aeration chamber, a final clarifier, and a chlorinator with a layout similar to the Whitewater System (see Figure 15.2). Clearstream's sewage treatment systems are available in sizes from 500 gallons per day to 1,000,000 gallons per day to serve residential, commercial, and industrial applications. Tables 13.1 and 13.2 show additional information provided from the manufacturer on this system.

MODEL	SERVES UP TO	RATED
Model 500N	5 Residents @ 100 G.P.P.	500 G.P.D.
Model 600N	6 Residents @ 100 G.P.P.	600 G.P.D.
Model 750N	7.5 Residents @ 100 G.P.P.	750 G.P.D.
Model 1000N	10 Residents @ 100 G.P.P.	1000 G.P.D.
Model 1500N	15 Residents @ 100 G.P.P.	1500 G.P.D.

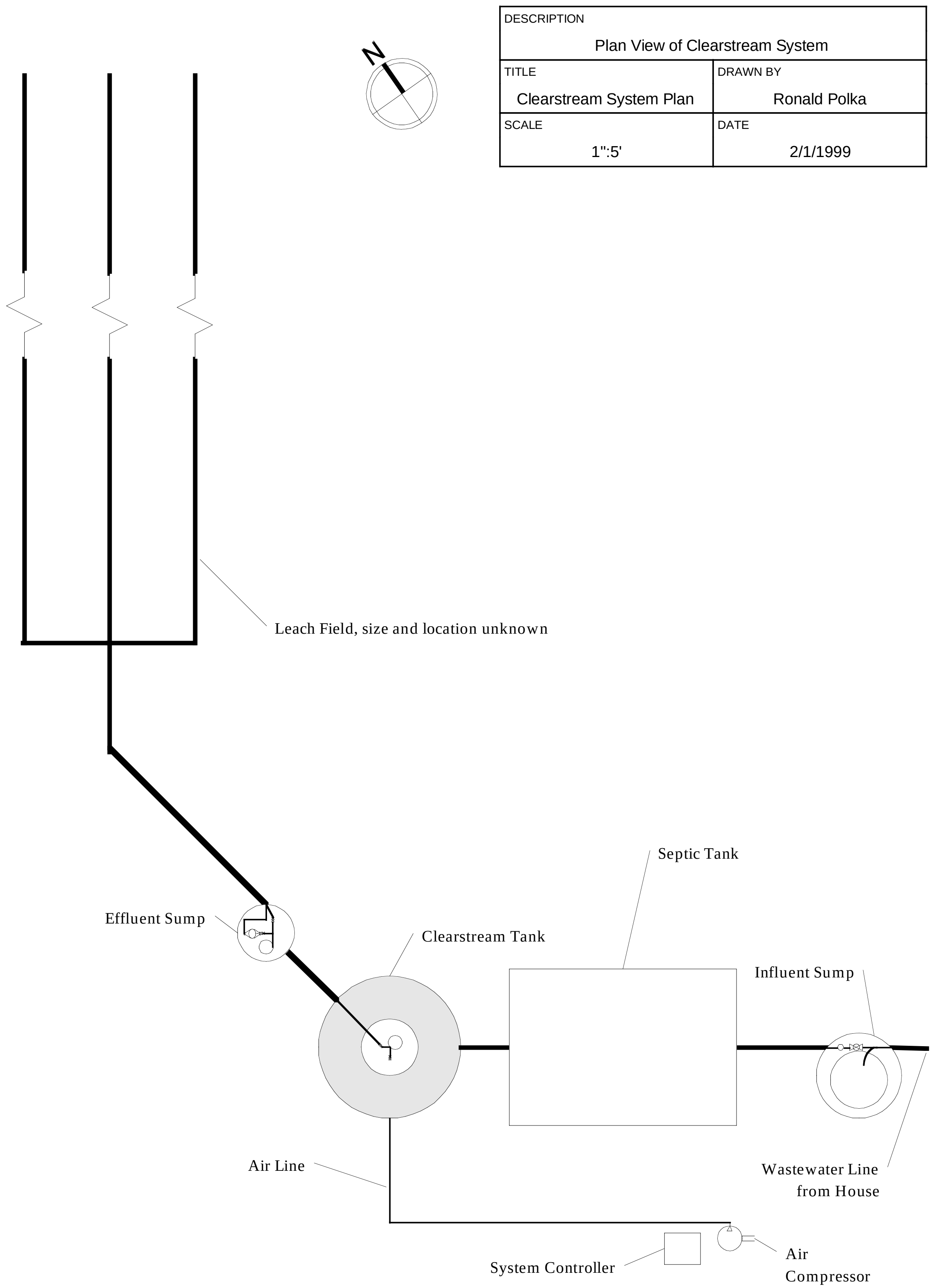

Figure 13.1. Clearstream System Plan View.

Table 13.1 Clearstream Onsite Wastewater Treatment Systems.

Model Number	Rated Capacity Gallons/Day	Classification
500N	500	CLASS I
500NC	500	CLASS I
600N	600	CLASS I
600NC	600	CLASS I
750N	750	CLASS I
750NC	750	CLASS I
1000N	1000	CLASS I
1000NC	1000	CLASS I
1500N	1500	CLASS I
1500NC	1500	CLASS I

With suffix C = Concrete Tanks
Without suffix C = Fiberglass Tanks

Table 13.2 Manufacturer Suggested Advantages of Clearstream Home Wastewater Treatment Systems.

TANK	AERATOR
Sturdy fiberglass or concrete construction	Long life
Easy to install design	Two-year warranty
Long-lasting non-corrosive components	Quiet operation
Easy to service without digging in yard	Very low electrical usage
Utilizes small pre-treatment tank for grease & trash removal	Installed outside tank to avoid flooding

Clearstream is listed by NSF and may sell products complying with all applicable requirements for certification nationally and internationally, but has advised NSF of distributors physically located in the following States:

Alabama	Alaska	Arizona	California
Florida	Georgia	Illinois	Iowa
Kentucky	Michigan	Missouri	New York
North Carolina	Oklahoma	South Carolina	Texas
Virginia	Washington	West Virginia	Wisconsin

Construction - Clearstream requires that a septic tank be utilized in front of the Clearstream tank. The septic tank is to keep grease, concentrated toxic household cleaners and other undesirable substances, that might upset the aerobic bacteria in the treatment tank, from getting into the system. Separating the heavy settleable material from the aerobic unit minimizes the maintenance costs. Clearstream manufactures treatment systems constructed of "Fiberglass Reinforced Plastic" (FRP) commonly called Fiberglass, and has Precast Reinforced Concrete tanks available. The Fiberglass tanks are constructed of chemical resistant isophthalic polyester resins and fiberglass reinforcement strands. The tanks are tested and certified by an approved International Association of Plumbing and Mechanical Officials (IAPMO) testing laboratory to meet and exceed the construction requirements of the IAPMO. Clearstream precast reinforced concrete tanks are designed by structural engineers specializing in precast design, and meet the appropriate ASTM (American Society for Testing Materials) "Specification for Precast Concrete Septic Tanks" (C1227-96), construction standards. Both the FRP and Concrete Clearstream tanks utilize non-corrosive hardware, made of PVC, Stainless Steel, Polyethylene or Neoprene.

Clearstream's aerators, control panels and accessory equipment are manufactured in the U.S.A. The aerator is housed in an attractive, protective enclosure that protects the aerator from the elements. The enclosure appears to be a plastic dog house. The Clearstream control panels utilize UL approved weather proof panel boxes designed for outdoor service.

13.3 Process Description

Clearstream wastewater treatment plants utilize an extended-aeration, activated sludge process. In the activated sludge process, microorganisms consume soluble contaminants that are in the wastewater. The microbes in the system utilize the organics as a source of food for growth and production of new microorganisms. The conversion of the organic matter from soluble to biological solids allows for removal of the organic matter by settling of the solids in the clarifier portion of the treatment process. The microbes that are settled in the clarification process are recycled back into the aeration portion of the process. This recycling of the microbes is what "activates" the process.

The organisms that treat the wastewater, are aerobic bacteria. Clearly, the transfer of oxygen into the wastewater by the aeration system is a critical component of the treatment process. In addition to adding oxygen to the process, the aeration system provides mixing to increase contact between the organic contaminants in the wastewater and the organisms that remove the contaminants. Interruption of the aeration system for a long period of time can have a serious impact on the process.

Extended aeration is a modification of the activated sludge process in which the microorganisms are allowed to remain in contact with the wastewater for long periods of time, usually greater than 18 hours. The long aeration period allows the organisms in the system to consume much of the organics in the wastewater and also to consume a portion of the microorganisms. Although this does not eliminate the need for removal of solids from the system, it does reduce the total amount of solids produced by the treatment process. Based on our previous discussion presented in the section on unit operations, one would expect this unit to remove BOD and to convert ammonia to nitrate. This system appears to have been modified by the installer, and is not an "off the shelf" product available from the manufacturer. There is a non-standard recycle pump, which returns nitrified water from the aeration basin back to the anoxic region in the trash or septic tank. The soluble carbon present and the anoxic conditions should provide denitrification. There is no evidence of this denitrification in the manufacturers' literature. In the event this system did provide excellent treatment, it is not clear that interested homeowners could purchase similar units. There also appears to have been modifications to the solid/liquid separation portion of the aerobic unit. There were fabric ribbons hanging in the clarifier area, which are not apparent on any literature from the manufacturer.

The manufacturer of the Clearstream system suggests that the effluent from the Clearstream System are of high enough quality to be used for subsurface drip irrigation of lawns. This option would also provide removal of the nitrate formed during the nitrification process.

13.4 System Installation

The Clearstream installation was not observed by the research team. Clearstream only sells its products to professional onsite installers. Clearstream not only offers initial factory training for installation and service of the equipment, but also provides on-going training and factory backup for problems encountered by wholesale customers. This is the extent of the installation detail provided by the manufacturer. It should be noted that this system was field modified by the manufacturer's representative. The system provided was not an "off the shelf" system. The project objectives specifically called for "off the shelf" hardware to be tested, and the modifications were not approved by project management.

13.5 System Operation and Maintenance

13.5.1 Manufacturer's Recommendations

According to the manufacturer, home aerobic treatment units, sand filters, mounds, chlorination units, lagoons, and other alternative or innovative onsite system technologies have different inspection requirements. Home aerobic treatment units usually come with maintenance service contracts that include regular inspections by a local manufacturer representative. Homeowners are sometimes required to renew these contracts after the initial two-year period, but should consider renewing them even if not required to do so. The Clearstream System has a two-year limited warranty against defects in material and workmanship from the date of purchase. The system and/or its components will be repaired or replaced with new or rebuilt equals to the original equipment, if all installation, operation and maintenance instructions of the manufacturer have been adhered to.

Service Schedule - In order for the Clearstream System to function at optimum performance levels, the system will require periodic service. The recommended service schedule is given in Table 13.3:

Table 13.3 Service Frequency for the Clearstream Wastewater Treatment System.

SERVICE	FREQUENCY
Repair on replace aerator	2 to 10 years
Clean filter on aerator	6 months to 2 years
Break up scum in clarifier	6 months to 2 years
Pump sludge from aeration tank	2 to 5 years
Pump sludge from trash trap	2 to 5 years
Check aeration diffuser	Annually
Check surge control weir	6 months

For the first two years from the date of purchase, your local installer, from whom you purchased your Clearstream System, will inspect your system on a routine basis for operational problems. Service on Clearstream electrical and mechanical components will be performed at no charge, if such components are found to be defective. The aerator is easily replaced in less than five minutes. The manufacturer claims the aerator will last many years and will be inexpensive to repair or replace. A Clearstream audio visual alarm panel will notify the homeowner immediately in the event of component or accessory malfunction.

To prevent malfunctions of your sewage system, the following manufacturer recommended guidelines should be followed:

- Any sewage treatment system, whether aerobic or septic, should not have inorganic materials (plastics, cigarette butts, condoms, throwaway diapers, etc.), which the bacteria cannot consume, discharged into the system.
- Large amounts of harsh chemicals, oil, grease, high sudsing detergents, discharge from water softeners, disinfectants or any other chemical or substance that kills bacteria should not be discharged into the system. Garbage disposals are not recommended.
- Excessive use of water, over the design flow of the system, will cause the system not to perform to its fullest capabilities.
- The proper operation of this or any other home sewage system depends upon proper organic loading and the life of the micro-organisms inside the system. Clearstream is not responsible for the in-field operation of a system, other than the mechanical and structural workings of the plant itself.

13.5.2 Observed Conditions

Of all the tested systems the Clearstream system was sampled the least, thus providing fewer opportunities for observations regarding system operation and maintenance. Early in the sampling program the residence was put up for sale. The last sample was collected on July 21, 1998. During the next sample visit it was noted that the house was unoccupied so no sampling was done. It remained on the market for the remainder of the study. Earlier in the summer it was noted that the homeowner was using the system effluent for surface irrigation of a lawn in the back yard. The system manufacturer recommends the use of system effluent for subsurface irrigation only. It is not known why the homeowner decided to apply the effluent directly to the lawn.

During sampling of this system it was observed that the wastewater line leading from the house to the septic tank always had water in it when it was opened for sampling. This indicates that the pipe and/or septic tank were not leveled correctly. It is not known whether this was caused by sloppy installation practices, or subsequent settling of the soil.

13.6 Reported System Performance

Data reported in this section come from NSF (1991) tests data. The Clearstream Model 500N plant tested has a rated capacity of 500 gallons per day (gpd). The plant has a 250 gallon pretreatment tank ahead of the aeration tank, providing for separation of heavy solids from the wastewater prior to introduction into the aeration tank. Wastewater enters the aeration chamber of the system, where it is mixed with organisms formed during decomposition of organic material in the wastewater. The mixture of the biomass and other solids in the aeration chamber is referred to as the mixed liquor. Mixing in the chamber is achieved by release of compressed air, near the bottom of the chamber through two porous media diffusers.

A conical shaped clarifier is located internal to the tank to provide settling of the solids from the mixed liquor. The mixed liquor passes into the clarifier by hydraulic displacement as wastewater enters the aeration chamber. Solids settled from the wastewater re-enter the aeration chamber.

Testing of the Clearstream Wastewater System, Inc. Model 500N Home Aeration Treatment System was conducted under the provisions of NSF Standard 40 for Individual Aerobic Wastewater Treatment Plants (July 1990 revision). NSF Standard 40 was developed by the NSF Joint Committee on Wastewater Technology. Clearstream test results averaged as low as 5 mg/l CBOD and 5 mg/l TSS. For Class I aerobic treatment system, the criteria for acceptance is an 85 percent reduction of Biochemical Oxygen Demand (BOD) and Total Suspended Solids (TSS). This standard does not require reduction in ammonia or in total nitrogen for Class I certification.

The performance evaluation was conducted at the NSF Wastewater Technology Test Facility in Chelsea, Michigan, using wastewater diverted from the Chelsea municipal wastewater collection system. The evaluation consisted of six months of testing, during which a seven week stress test was conducted. The evaluation consisted of three weeks of dosing without sampling to allow for plant start-up, sixteen weeks of dosing at design flow, seven weeks of stress test and five weeks of dosing at design flow. Sampling started in the fall and continued through the winter and into spring, covering a full range of operating temperatures.

Standard 40, in Section H, provides for exclusion of up to ten percent of the effluent sample days, not to exceed one during stress testing, in completing the pass/fail determination. Excluding four sample days with high effluent BOD_5 and suspended solids concentrations, the average effluent BOD_5 was 6 mg/L, ranging between <5 and 17 mg/L, and the average effluent suspended solids was 8 mg/L, ranging between <5 and 57 mg/L. Over the course of the evaluation, the Clearstream 500N produced an effluent with BOD_5 ranging from <5 to 570 mg/L, suspended solids ranging from <5 to 3,100 mg/L, and pH ranging from 7.4 to 8.1.

The Clearstream 500N produced an effluent that successfully met the performance requirements established by NSF Standard 40 for Class I effluent. The maximum arithmetic mean of seven consecutive sample days was 13 mg/L for BOD_5 and 28 mg/L for suspended solids, both well below the allowed maximum of 45 mg/L. The maximum arithmetic mean of 30 consecutive sample days was 8 mg/L for BOD_5 and 13 mg/L from 95 to 97 percent for BOD_5 and 93 to 98 percent for suspended solids, consistently above the requirement of 85 percent. The effluent pH during the entire evaluation ranged between 7.4 to 8.1, within the required range of 6.0 to 9.0. The Clearstream 500N also met the requirements for noise levels (less than 60 dba at a distance of 20 feet) color, threshold odor, oily film and foam.

13.7 Field Trial Results

Flow characterization and hydraulic analysis of the system was not performed. Sampling was limited because the owner moved out during the early part of the study and the house was not occupied through the remainder of the study.

13.7.1 Flow Characterization

An influent flow characterization was not performed on this system

13.7.2 Hydraulic Analysis

A hydraulic analysis was not performed on this system

13.7.3 Water Quality Analysis

The Clearstream system was scheduled to be performance tested for 54 weeks. Unfortunately, the sampling team responsible for the first 19 weeks of sampling misunderstood the sampling protocol, and none of the data sets contained paired samples. The combined NMSU/UNM sampling team assumed responsibility for the system in week 20. By week 22 the sampling sumps had been rebuild to specification, and sampling commenced. The Clearstream system was performance tested for 13 weeks from the April 29, 1998 through July 21, 1998 with 7 influent and effluent samples collected. There are only two sets of data for this system that the investigators consider completely reliable. Unfortunately, after 6 weeks of sampling (3 sets of samples) the residents unexpectedly went on vacation. Of the three sample sets collected prior to the vacation vacancy, one set was lost due to lab problems. For the four weeks following the vacation vacancy, there were pump and flow meter problems, which compromised the data. The week the pump and flow meter problems were corrected, the residents moved out permanently. It is believed that the residence is still vacant at this point in time (3/8/99). There are only 2 sets of data for this system that the investigators consider completely reliable.

Because of the resident's work schedules, the residents were seldom on the site when the sampling teams were present. The residence vacancy was deduced by reviewing the sampling data collected from the site, and then confirmed. The average daily flow recorded by the onsite flow meter indicated as shown in Figure 13.2, that flow varied between 16 and 147 gpd over this period. The recorded 16 gpd flow is believed to be a result of homeowner intervention. The average daily flow for the preceding two sample periods is 129.5 gpd. A reduction to 12 gpd indicates that no flow was recorded during most of the two-week period between samples. On May 28, 1998, it was noted that a garden hose was connected to the effluent pump outlet pipe and was being used to water the lawn. This action by the homeowner bypassed the water meter and caused an incorrect reading of water use for the preceding two week period. On June 10, 1998, two valves were plumbed into the effluent sump to enable the homeowner to water his lawn without compromising the flow data. On June 24, 1998, no overnight flow into the effluent sump occurred. It was presumed that the family was on vacation at this time. On July 8 and July 22, 1998, samples were taken. During the next site visit on August 5, 1998, no overnight flow into the effluent sump occurred. In addition, during the site visit on August 5, 1998, it was noted that the house was vacant. At this time, it was presumed that the residence had been sold and the

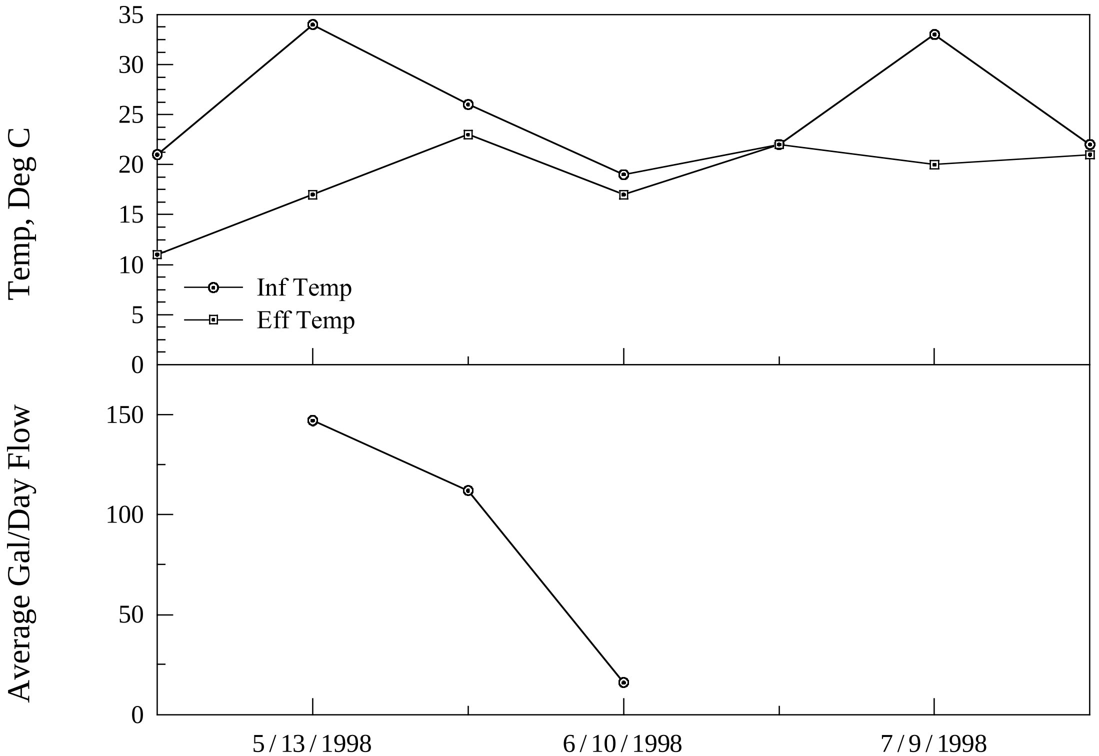

Figure 13.2. Clearstream System Experimental Data.

owners had moved. Sometime after the June 10 plumbing repairs the effluent pump stopped working for unknown reasons. From that time on, the effluent sump filled with water and flowed by gravity to the leachfield. An attempt to find the problem was made but was unsuccessful because the pump and float switch, were hardwired to the house. This lack of access to the pump electrical circuit prevented pump troubleshooting and replacement.

Temperature data (Figure 13.2) for the site indicated no significant difference between the influent and effluent with mean values of 25.9 and 18.7 °C, respectively. Effluent temperature, reflecting the actual operating temperature of the process, varied from 11 to 23 °C over the study period. These temperature differences can affect the performance of biological treatment process particularly nitrification, which requires a temperature greater than 15 °C to function effectively. If further testing is to be done on this system, it should be timed so that the system has 4 weeks of warm effluent temperatures to acclimate the nitrifiers. Then at least 6 warm weather and 6 cold weather samples should be collected. This will indicate the nitrifying capability of the system on all conditions with an established culture.

Data for EC, chlorides, sulfates, and ortho-phosporous are shown in Figure 13.3. Figure 13.4 shows TSS, COD, DO, and pH. This data, with the exception of DO, is not discussed in detail because of the sparse database.

The BOD_5 values for this system were not graphed in part because only half the values were provided by the laboratory after samples had been submitted. Many of these were reported as over range or under range and were not usable. The BOD_5 values for the influent had a mean of 218.2 mg/L. The BOD_5 values for the effluent had a mean of 49 mg/L. This data is encouraging, but because of the very small data set, the statistical significance of the means is suspect.

The mean flowrate and the mean BOD were used to calculate a mass BOD loading for each of the systems. All of the other systems, with BOD measured prior to the septic tank, showed nearly identical BOD mass with 0.5 lbs/day going into the systems. This system had a BOD mass loading of 0.08 lbs/day going into the system. Another indication that the data set had problems.

The DO data values for the system shown in Figure 13.4 indicated variation influent DO with concentration ranging from below 0.5 mg/L to over 3.1 mg/L. The effluent values averaged 2.53 mg/L with a standard deviation of 1.66. The effluent DO was in the appropriate range for good biological treatment. It is believed that this unit would be significantly improved if the solid liquid separation characteristics were improved.

The nitrogen species data, shown in Figure 13.5, had so few data points that it is not wise to draw any conclusions from the statistical analysis. The Whitewater system showed a 72% removal of TN during the performance period with the best conditions for nitrogen removal. There is every reason to believe that the Clearstream system should be capable of at least that level of performance. However, the TN values for the Clearstream system indicate an 18% removal. Because of the lack of continuous data record, it was not possible to identify the point at which nitrification started to occur and eliminate data outside of that region. There is simply not enough reliable data to evaluate this system’s nitrogen reduction capability.

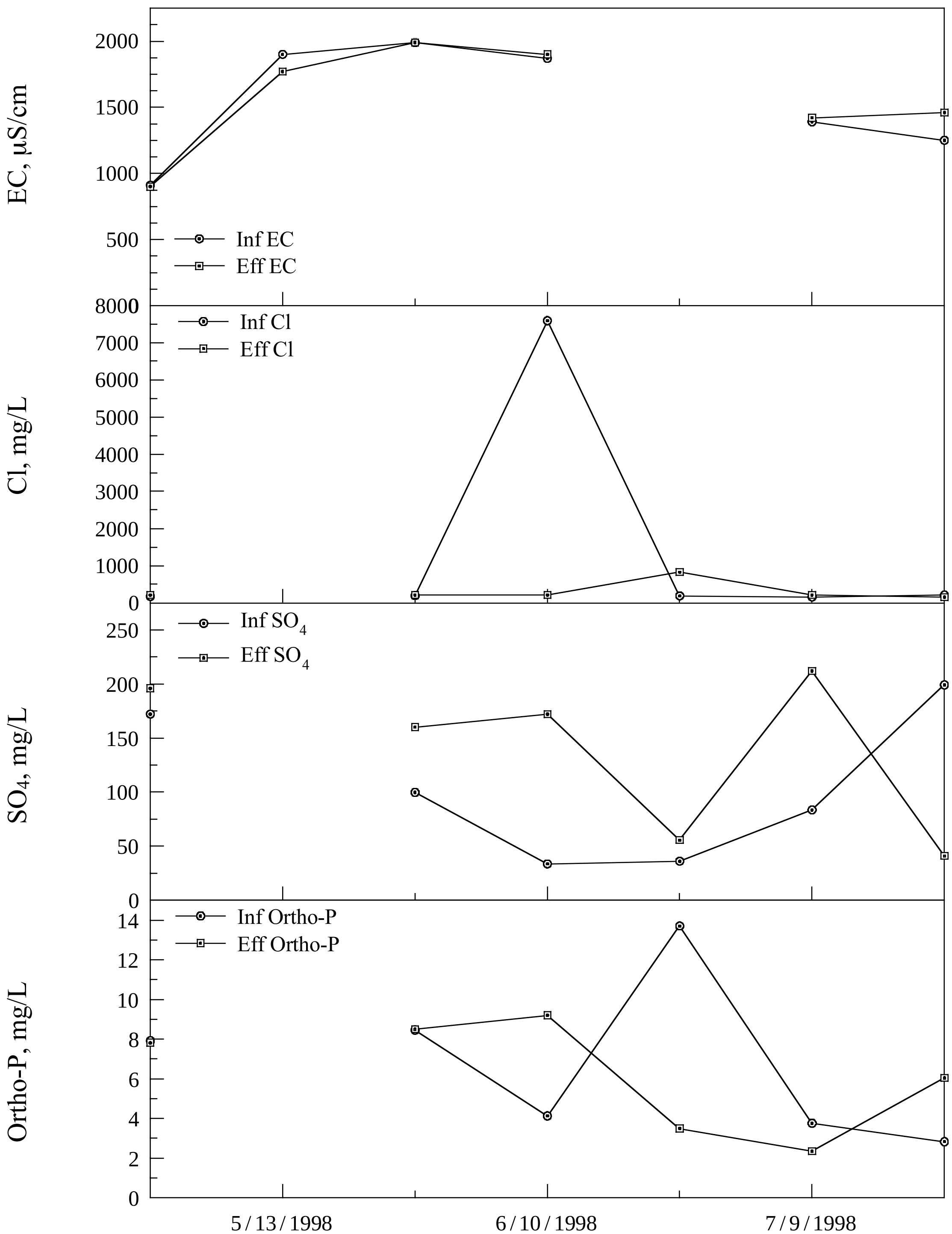

Figure 13.3. Clearstream System Experimental Data.

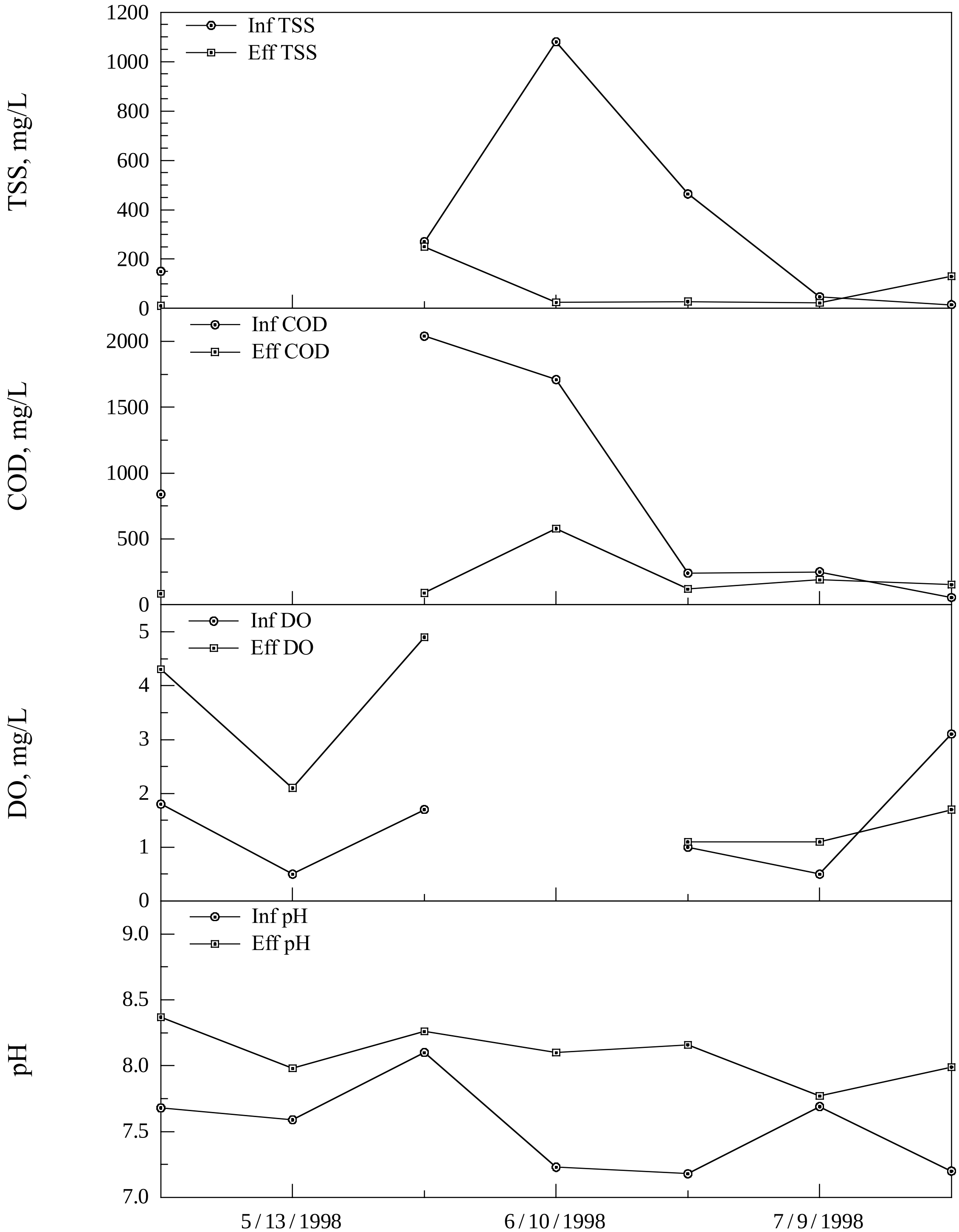

Figure 13.4. Clearstream System Experimental Data.

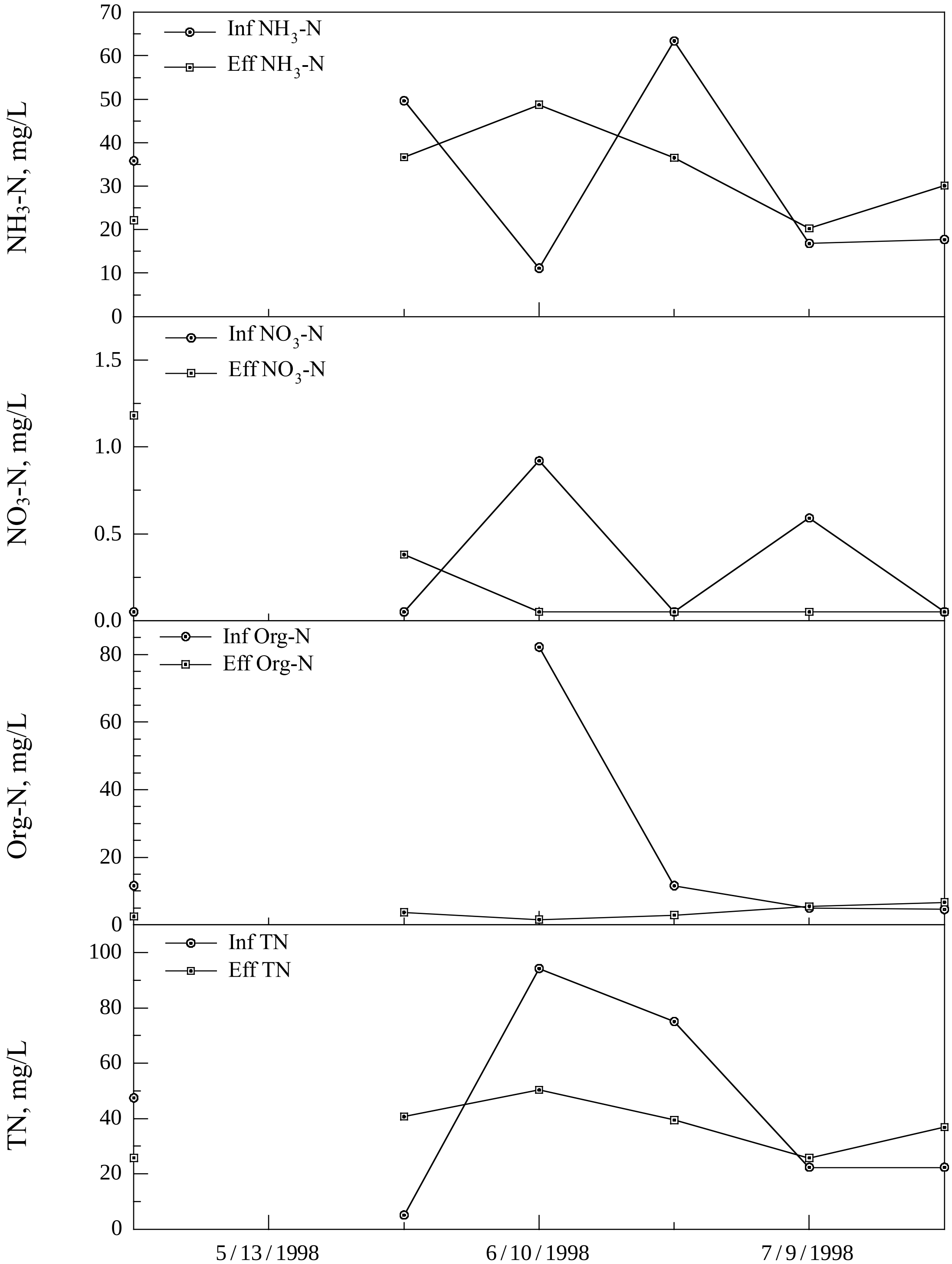

Figure 13.5. Clearstream System Experimental Data.

The fecal coliform (FC) data for the septic system was highly variable with mean influent and effluent values of 1.09 x10^5 and 8.0x10^3 cfu/100mL, respectively. If the lowest value is excluded, this resulted in a removal rate of 98.0 percent, which is almost 2 log removal. Influent values ranged from 200 to 2.0 x10^5 and effluent ranged from 930 to 2.6 x10^4 cfu/100mL.

13.8 Conclusions

The Clearstream system was scheduled to be performance tested for 54 weeks. Unfortunately, the sampling team responsible for the first 19 weeks of sampling misunderstood the sampling protocol, and none of the data sets contained paired samples. The replacement sampling team, which was a combined NMSU/UNM team, assumed responsibility for the system in week 20. By week 22 the sampling sumps had been rebuilt to specification, and sampling commenced. The Clearstream system was performance tested for 13 weeks from the April 29, 1998 through July 21, 1998 with 7 influent and effluent samples collected. There are only 2 sets of data for this system that the investigators consider completely reliable. The flow characterization and reactor tracer analysis were not performed on the system. In addition, the installation, maintenance, and operation of the system were not evaluated. The system as tested evaluated performance based on effluent from the house as the influent to the system. The Septic Tank is considered part of the Clearstream system.

Observations regarding system operation and maintenance indicated that the manufacturer supplied information on the off the shelf unit was well documented and was transferred to the installer and the homeowner. However, the modifications incorporated by the installer where completely undocumented. It was very difficult to tell exactly what had been done to the system. This is especially important considering the number of times the system changed hands during the course of the study.

There are some striking similarities between the geometry of this system and the geometry of the Whitewater system. It is expected that under similar installation conditions they should behave very much alike. There was reason to believe that the Clearstream system, as installed, should have significantly outperformed the Whitewater system. The Clearstream system has a trash tank, which is not included in the Whitewater system, upstream from the aerobic unit. More importantly, the Clearstream system was not a stock system, but was modified in two areas. First it had a pump added which recirculates nitrified effluent to the anoxic trash tank for denitrification. This recirculation loop was not stock on the Clearstream and was not included in the Whitewater system, which was a stock “off the shelf” system. In addition to the recirculation loop the Clearstream had a fabric of some sort hanging in the clarifier. It appeared that this modification from the stock system was intended to improve the solids liquid separation in the aerobic process. Inspite of this modification, the Clearstream system seemed to suffer from deficiencies in the solid/liquid separation area. Unfortunately, because of occupancy problems, the data set was not adequate to demonstrate the capability of this system; either poor or good. This system really needs another 6 months of good field data collection. We were aware when we decided to test these systems at occupied residences that a situation like this might develop. Unfortunately there can be no contingency plan for a resident unexpectedly moving out.

Chapter 14 - FAST System

14.1 Site Description

The system tested, was a Bio-Microbics Aerobic Treatment Unit. This system is characterized as a fixed film/suspended growth biological (FF/SGB) treatment system designed to treat 500 gallons per day of domestic wastewater. A manufacturer representative located in Las Cruces, New Mexico provided the unit and an approved local onsite system installer from Bernalillo installed the unit. The test site was a three-bedroom house located in Bernalillo County, east of the Sandia Mountains in a fractured bedrock region. The house was estimated to have three full time occupants, has a water softener for a portion of the water flow, and a garbage grinder is installed. The water softener backwash at this residence does flow to the onsite system. Because the installation was in an area with a shallow fractured bedrock a mound leachfield system was required for effluent disposal. The exact size and location of the leachfield are unknown. The system layout with sample sumps and required setbacks is shown in Figure 14.1. Sampling was set up using methods outlined in the previous methods section.

14.2 System Description

The system as shown in Figure 14.2 is a pre-engineered modular unit designed to treat wastewater for residential, commercial, high strength, and small community applications, with a fixed film aerated system utilizing a combination of attached and suspended growth. A typical FAST system consists of a pretreatment tank (trash tank/septic tank), an aeration chamber with a media suitable for growth of a fixed film of microbes, and a final clarifier. Installation of the lightweight and durable FAST system is easy. It simply mounts into a septic tank. The systems are available in sizes from 500 to 1,000,000 gallons per day to serve residential, commercial, and industrial applications. This system is capable of nitrification/denitrification in a single tank without any system modifications. The system consistently reduces nitrogen levels-including nitrates and all other nitrogen species by over 70%. This system combines the stability of fixed film media and the effectiveness of proven activated sludge treatment, making the FAST system technologically advanced and extraordinarily reliable.

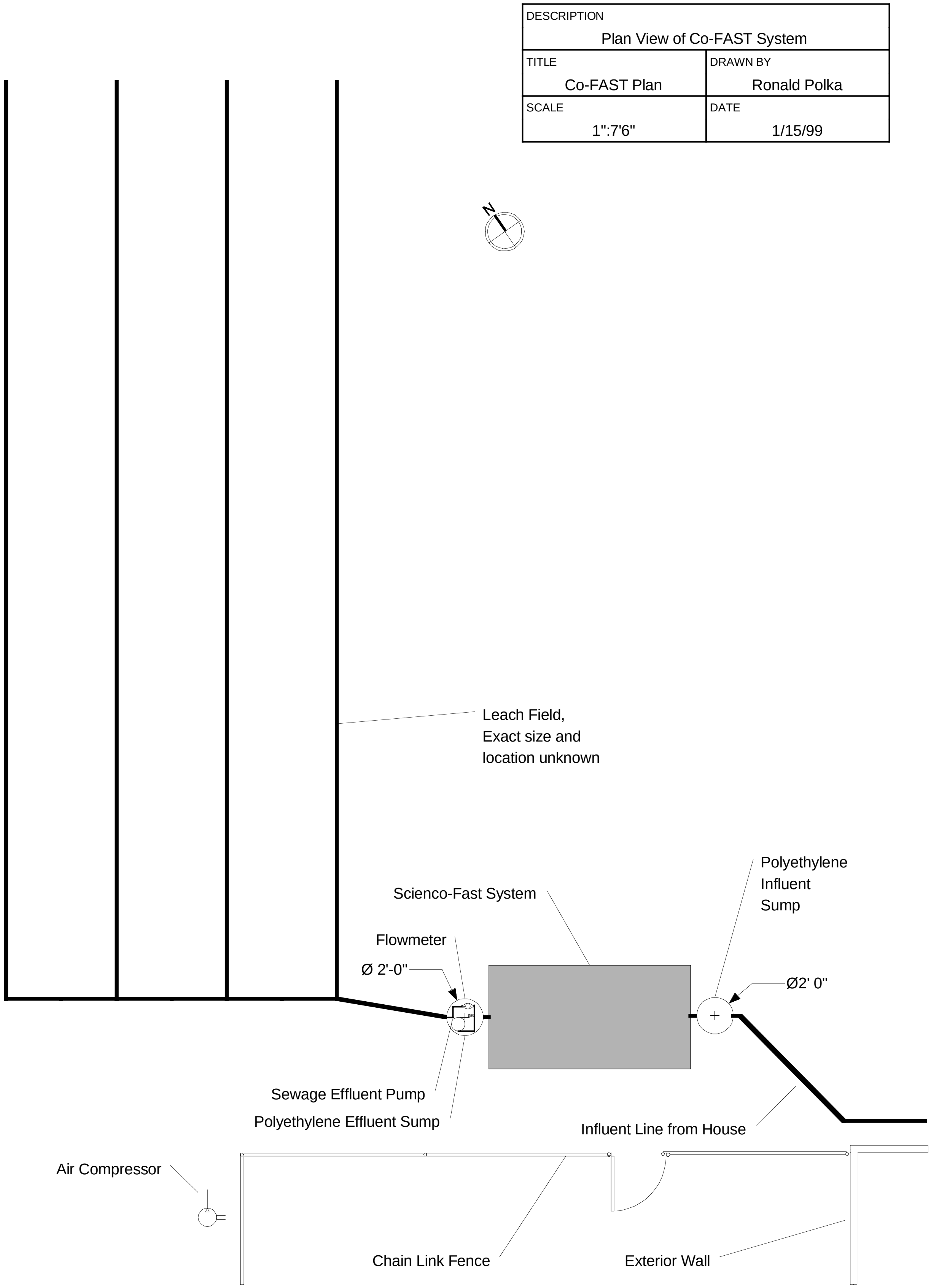

Figure 14.1. Co-FAST System Plan View.

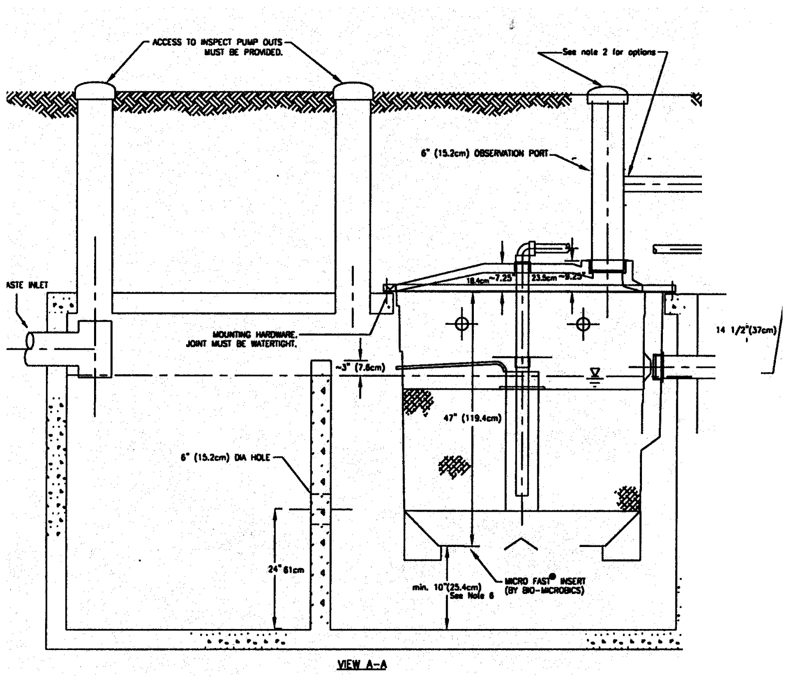

Figure 14.2. Side View of the FAST Onsite Wastewater Treatment System.

A FAST system provides an ideal home for large volumes of friendly organisms in the inner aerated media chamber to digest the wastewater and turn it into a clear, odorless, high-quality effluent. The attached growth system assures that more organisms remain inside the system instead of being flushed out, even during times of peak hydraulic flows (for example, during large social gatherings or on multiple-washload laundry days). During times of low usage, the large volumes of thriving organisms prevent a dying-off of the system, making the FAST system equally well suited to intermittent use applications.

14.3 Process Description

The science behind a FAST wastewater treatment system is environmentally sound and simple. In a traditional septic tank and in some of the aerobic treatment systems, the biomass is suspended in the wastewater. When the biomass is suspended in the water, it has a greater opportunity to be discharged into the drain field. The Fixed Activated Sludge Treatment system keeps the active biomass on the media and not in the water. This allows for cleaner water to be discharged to the drain field. FAST is an acronym for Fixed Activated Sludge Treatment.

The principal treatment unit in this system is a proprietary unit known as the Bio-Microbics FAST® aerobic unit. The FAST wastewater treatment plant utilizes an extended aeration activated sludge process. In the activated sludge process combined with a fixed film aerobic process microorganisms, either in suspension or attached to the honeycomb media, consume soluble contaminants which are in the wastewater. The wastewater is recirculated across the honeycomb media using an airlift pump. Air is supplied to the bottom of a draft tube placed in the center of the media. The air lifts the wastewater from the bottom of the reactor and splashes it across the top of the media. It then recirculates down through the media and the process begins again. The microbes in the system utilize the organics as a source of food for growth and production of new microorganisms. The conversion of the organic matter from soluble to biological solids allows for removal of the organic matter by settling of the solids in the clarifier portion of the treatment process. The microbes that are settled in the clarification process are recycled back into the aeration portion of the process. This recycling of the microbes is what "activates" the process.

The aeration process also provides circulation of wastewater to increase contact with aerobic bacteria. Also, anaerobic zones within the FAST chamber result in denitrification. This system provides a nitrified effluent prior to discharge. It has been suggested that denitrification can be improved in this system by adding a separate anaerobic biofilter (ABF) into the treatment train following the FAST unit. The FAST treatment tank is separated into two chambers. The first chamber receives raw wastewater influent and provides primary treatment. The first chamber, functioning much like a normal septic tank, is utilized in front of the FAST tank, to keep grease, toxic household cleaners and other undesirable substances, which might upset the aerobic bacteria in the treatment tank, from getting into the system. Separating the heavy settleable material from the aerobic unit minimizes the maintenance costs. Wastewater overflows via a 6-inch diameter orifice in the partition wall to a second chamber where secondary treatment is provided by the FAST unit. A blower mounted outside on top of the treatment tank provides the air source for the FAST aeration. The FAST process consists of the treatment tank and the blower (air source). In the FAST process, bacteria called the biomass break down biodegradable waste into carbon dioxide and water. The process occurs continuously as long as the biomass is supplied with food (incoming waste) and oxygen (air) in a suitable environment. The blower provides continuous air to the treatment tank through the air supply pipe. The air supply pipe combines with the draft tube to create an air lift. This air lift is the means by which air and wastewater are mixed within the tank. The air lift lifts the wastewater to the splash plate. The wastewater is cascaded off the splash plate across the surface of the honeycomb media. The honeycomb media is the heart of the FAST process and is suspended in the septic tank. The media contains the biomass, the bacteria that stabilizes the wastewater. By growing on the honeycomb media and receiving food and air necessary for growth from the airlift, the biomass is allowed to stabilize (eat) the waste before it is discharged to the drain. Eventually, the biomass dies, sloughs off the media and collects at the bottom of the tank. The solid material that the biomass cannot process, which includes portions of the bacteria that die and settle in the septic tank, must be removed periodically by normal pump-out removal.

14.4 System Installation

According to the manufacturer, the FAST systems may be located in the same position relative to the house and water supply as any conventional septic system. The manufacturer recommends some basic guidelines should be followed:

- The FAST system is only designed to withstand the weight of the soil up to a burial depth of 4 feet (1.2 meters). It is not designed to withstand loads from concrete slabs, vehicles, or buildings. Do not place the tank in a location where it could be subjected to additional weight. Vehicles weighing more than 2,500 pounds per wheel, approximately a fully loaded 3/4 ton pickup truck, should not be driven in the area to minimize the risk of damage to the septic tank and associated piping. If the burial depth must be more than four feet or if the area is subjected to additional weight, such as from occasional standing water, check with the manufacturer before proceeding.

- The FAST system must be located so that sufficient slope is provided for the influent and effluent lines. If either of these two lines becomes blocked, there is risk of excess water backing up into the house. A 2% slope is recommended for this. A 2% slope equates to a drop of two feet over a run length of one hundred feet. This also equates to 1/4 inch per foot.

- The FAST system must be located so that vents and air intakes will be protected from snow drifts. The vent pipe allows for venting of air and non-harmful carbon dioxide created by the process.

- Avoid locating the FAST system in high groundwater areas where the tank could possibly float up and become dislodged.

- The blower housing should be no more than 100 feet from the FAST system. The blower must be placed at an elevation higher that the flood plain.

Documentation must be maintained by installers, verifying the minimum dimensions of the tank, as well as its structural integrity. If the tank is out of the minimum dimensional range specified, the FAST system will not operate properly. The effluent quality could suffer and may not meet the standards. The performance of this unit depends on the disposal method of the effluent. The method and arrangement for disposal must not cause a backup or any other interference with the treatment plant's operation. The technique and equipment used for the effluent disposal must be approved by the local or state health and environmental agencies.

Before installation of the module, check the tank to ensure it is level within 1 inch from inlet to outlet and 1- 1/2 inch from side to side. When installing a new septic tank, make sure the inlet is a minimum of 2 inches (5.08 cm) above the outlet. Once the tank is in place and level and in compliance with local health, environmental and plumbing regulatory agencies, the installation may begin.

14.4.1 General Assembly

- Insert 4" factory provided gasket into effluent bole on module liner. (Be sure to push all gaskets in until gasket flange meets with module surface.)

- Place module liner through hole in top of tank. Place module lid on top of module liner, being careful to line up air line hole in module lid with coupling at top of draft tube inside module insert, and drill holes for anchoring module to tank using pre-formed holes in module lid.

- Remove module lid and lift module liner to apply sealant between mating surface of module liner and top of tank (any readily available, non-hardening sealant on the job site that will provide a water-tight seal is appropriate)

- Apply sealant to mating surface between module liner and module lid. Position module lid on top of module liner and secure module lid and liner to tank, using holes drilled in step 2 and commercially available screws and/or anchoring system.

- Bevel and soap the end of 4" Sch 40 PVC pipe to be used as effluent/outlet line. Insert the line through side of tank and into 4" gasket in the outlet hole on the module. Push the pipe in 2" until it stops. Do not use excessive force when inserting the outlet pipe into the FAST Module. This will provide a water-tight seal around the outlet pipe on the FAST module. You will also need to provide a water-tight seal around the outlet pipe where it exits the tank.

- Cut a piece of 2" Sch 40 PVC pipe, to be used as the air line entering module lid and insert, to the desired length (or longer). Bevel and soap the end of 2" pipe. Insert the 2" factory provided gasket into the air line hole in the module lid. Insert the beveled and soaped end of the pipe through the module lid. Using the 6" hole in module lid, reach inside the module and thoroughly clean all pipe soap from the lower 6" of air line or leave cover up off liner to have access to clean and glue the pipe. When this process is complete, slide the cover down the pipe to the liner. Using PVC primer and glue, secure the air line into coupling at top of draft tube in module insert. If cover was left up, slide it down on the pipe to the liner.

- Bevel and soap the end of a 6" Sch 40 PVC pipe to be used as observation port/vent. Insert 6" factory provided gasket into observation port access hole in module lid. Insert pipe until it stops, which should be - 2" inside module lid. Do not push all of the way down to the media surface.

- Pressure switch installation procedure. Drill appropriate sized hole for 1/8" nipple in the blower outflow pipe. Insert pressure switch into hole, nipple first, and glue into place. Connect alarm box lead to the "Common lead on the pressure switch". Run these leads back to the alarm unit with the blower wiring.

- Always check with local utility companies for the location of water, and gas lines and electricity and telephone cables, or any hazards below grade prior to excavation. Failure to do so may result in severe injury or death.

14.4.2 Blower Assembly

- Remove power from the blower assembly by switching the circuit breaker in the FAST system control panel to the OFF position. Also, make sure to switch off the circuit breaker in the building's main service panel. If the main circuit breaker to the building blower is to be disconnected for more than 48 hours, it may be necessary to prevent the discharge of wastewater into the drain field. All electrical work should be performed by a qualified electrician and per all applicable electrical codes.

- Remove blower housing cover by unscrewing the blower housing cover mounting bolts and lifting the lid off the blower housing base.

- Remove the motor conduit box cover on the blower motor by unscrewing the Screw(s) securing it to the conduit box. Check with an appropriate measuring device to determine if there is power to the electrical wire leads in the conduit box before proceeding.

- If there is no power at the wire leads, disconnect the power leads from the motor leads noting the connections for proper reconnection during installation. Insulate and support the wires out of the way of the blower so they won't interfere with the blower removal process.

- Disconnect the outlet piping of the blower either by disconnecting the union (if used), unscrewing the pipe from the blower, or cutting a section of the outlet piping. If the piping needs to be cut, be sure to cut the pipe in an area such that a coupling or union (preferred) can be installed at the cut when the blower is re-installed.

- Cover the openings in the pipe where the separation has occurred to prevent any foreign material from entering the piping.

- Remove the mounting bolts securing the blower flange to the housing base. (May need to remove blower base from concrete pad). Lift the blower assembly off the Blower housing base.

- Make sure the circuit breaker in the FAST system control panel and the main circuit breaker to the building are in the OFF position. Set the blower assembly on the blower housing base. Attach the blower flange to the blower housing base. Connect the blower outlet piping to the air line by connecting the union (if used), screwing the air line into the blower, or installing a coupling at the cut, depending on the method of removal.

- Check the power leads coming into the blower housing with an appropriate measuring device to determine if there is power at the leads. If there is no power at the leads, connect the leads to the blower using the correct scheme as noted on the inside of the motor conduit box cover or name plate. Insulate the wires and fit them inside the conduit box in a professional manner.

- Attach the conduit box cover to the conduit box using the two screws removed during the removal procedure.

- Test the blower for correct operation by switching the circuit breakers in the control panel and the building to the ON position.

- Put the blower housing cover on the blower housing base by matching the cover bolt holes, with the base bolt holes. Bolt the cover to the base using the bolts removed during the removal procedure.

Final installation inspection: It is the responsibility of the installer to fill the tank to operating level prior to backfilling the excavation. If the tank is not filled, heavy rains after backfilling, could cause the tank to float and damage the surrounding grounds.

Before the tank excavation is backfilled:

- Fill the tank to normal operating level and check for leaks in all water-tight seals. If leaks are found reseal those areas.

- Ensure that the air line is properly installed and connected to the tank and blower. Turn on the blower and observe the operation of the airlift through the observation port. If the unit is level, has no leaks, and has even flow dispersion of the water, then backfill the excavation.

14.5 System Operation and Maintenance

The manufacturer's guidelines are provided, as well as our observations in this section.

14.5.1 Manufacturer's Recommendations

- To prevent malfunctions of your sewage system, the following guidelines should be followed Any sewage treatment system, whether aerobic or septic, should not have inorganic materials (plastics, cigarette butts, condoms, throwaway diapers, etc.), which the bacteria cannot consume, discharged into the system.
- Large amounts of harsh chemicals, oil, grease, high sudsing detergents, discharge from water softeners, disinfectants or any other chemical or substance that kills bacteria should not be discharged into the system. Garbage disposals are not recommended.
- Excessive use of water, over the design flow of the system, will cause the system not to perform to its fullest capabilities.
- The proper operation of this or any other home sewage system depends upon proper organic loading and the life of the micro-organisms inside the system. FAST is not responsible for the in-field operation of a system, other than the mechanical and structural workings of the plant itself.

If the property is going to be used seasonally and shut down completely for an extended period of time (i.e. summer use only and then abandoned for the winter), we suggest that the blower is also shut down. The blower should be restarted upon return to the property. Your local service provider may be contacted to perform these functions (It is also possible to arrange for the re-starting of the blower a week or two in advance of return through your local service provider). If the property will be used weekends only, it is best to leave the blower on continuously through out the season of use until an extended period of absence is anticipated (Extended period being at least 5 weeks or more).

The FAST wastewater treatment system operates automatically and continuously. There are no operating procedures for the user of the FAST wastewater treatment system to perform. However, as with any home appliance or equipment, simple periodic checks should and can be made to aid in the prevention of costly repair problems. Generally, the FAST wastewater treatment system can be checked by sight and by smell.

The FAST wastewater treatment system is an odor-free system. Therefore, there should be no septic smell emanating from your system. Should there be a sulfurous “septic” smell associated with the FAST system, contact your Bio-Microbics service technician. Wastewater backup is characterized by wastewater flowing back into the house or slow movement of wastewater in the drains. This may indicate a problem with your FAST wastewater treatment system unit. Identify where the backup is occurring within your home’s plumbing system. If no material is blocking the drain, contact your Bio-Microbics service technician.

DANGER: DO NOT attempt to service any components of the FAST yourself, call your Bio-Microbics service technician. Potentially hazardous gases and waste matter are contained in the treatment tank and only trained, certified service technicians are authorized to service your unit. Servicing by unauthorized personnel may result in death or bodily injury.

The FAST wastewater treatment system operates automatically and continuously. However, some routine preventive maintenance should be performed to ensure a long, reliable life of the plant. The maintenance procedures for the user of the FAST wastewater treatment system include keeping the vents and the blower housing clear of debris. The homeowner should monitor the status of the system alarm indicator light, substances introduced into the system, and the frequency of required pump out as determined by the service provider. The operational procedures for the FAST wastewater treatment system are minimal. Normal operation of the unit requires continuous operation of the blower and regular discharge of wastewater to the unit. Leaves, snow, or other material must not be allowed to block the blower intake. The following items should be included in a regular preventive Maintenance schedule.

As needed clean the screen, covering the vent pipe, or around 1/2" vent holes if vent option A is used, and the screens located on the blower housing. If these are left unchecked, there is a risk of the blower becoming starved and of blower damage should this condition develop. Blockage of the air inlet or vent could also reduce the efficiency of the treatment process if the oxygen is not allowed to replenish for aeration.

Annually-Check and clean the blower inlet filter when it is dirty. If this is left unchecked, damage to the blower may result and treatment quality may also suffer. Remove the nut on top of the filter and lift off the cover. The filter element is inside. If it is necessary, replace it with a new one if it cannot be cleaned. Check for vibration and the amperage draw of the blower to be sure it is within acceptable limits as noted on the blower nameplate.

As Required by Measurement of Sludge Depth - As the FAST system processes the raw domestic waste, sludge and sloughed-off bacteria will collect on the bottom of the vessel. This will have to be pumped out periodically. This time interval will change with changing load conditions. The time interval is also dependent on the size of the vessel.

To accurately determine the sludge depth, open up the pump out cover to the primary zone and insert a sludge measuring instrument and take samples. If the sludge depth in the primary zone is greater than 20 inches, it is necessary to pump the unit down. The sludge depth of the secondary zone (which contains the FAST system) must also be checked. Open up the pump out cover to the secondary zone and measure the sludge depth. If the sludge depth in the secondary zone is greater than 14 inches, it is necessary to pump the unit down. Always pump out both sections of the unit even though only one zone may require it.

Avoid pumping the unit down after periods of heavy rain or when the groundwater is likely to be above the bottom of the concrete tanks. Emptying the tank under these conditions could cause the tank to float up and become dislodged. Open the pump out cover and insert the hose. Be sure to pump out both sections of the reactor. Once the unit has been pumped out, immediately refill the tank with clean water to reduce the risk of the tank floating. Close the pump out cover making sure it is watertight. The disposal of the solids that have been removed must comply with local and state regulations.

During service calls, the authorized service person will check the blower for proper operation and perform preventative maintenance including cleaning of the blower intake and inspection of control panel light. The service provider will also measure the solids level in the septic tank and recommend pump out when necessary.

An extended service policy is available and may be purchased through your local Bio-Microbics distributor. The extended service policy meets the local inspection requirements but as a minimum should include one service call per year. The extended service policy may include cleaning of the blower intake, inspection of control panel light and recommendations on pump out when necessary.

The FAST system is furnished with an alarm circuit that will monitor the mechanical aeration components. If the blower should fail, the lamp will flash and the audible alarm will sound. If the blower should fail and trip the circuit breaker, a relay will then sense no load to the motor and to go into the alarm mode. If a high water condition exists, the alarm will activate.

This section is a summary of the different types of failures that are the most likely to occur in the Bio-Microbics, Inc. FAST system. The consequences of, and the steps taken to prevent these failures are also explained. Several types of failures can occur in a unit with the wide variety of components and systems present in this plant. Mechanical, electrical and process failures are the predominant concerns. Some components are subjected to more than one type of failure. Any mechanical or electrical failure will result in a process failure.

Mechanical failure of blower: The prime opportunity for failure of the blower is the internal bearings. They can fail from lack of lubricant or contaminated lubricant. Another opportunity for failure is excessive wear of the impeller resulting in lower volumes of air delivery. Electrical failure may take the form of overheating or shorting out because of moisture or dirt. Both of these modes of failure have been addressed by using a TEFC motor. With the motor being totally enclosed, the problem of dirt and moisture collecting on the windings to shorten insulation life has been eliminated. The fan cooling will help the motor maintain allowable running temperature. The totally enclosed rating helps maintain the internal cleanliness of the motor.

Protective inlet screens have been located on each end of the blower housing. If one screen becomes blocked by debris, the opposite screen should still be sufficient. The suggested routine preventive maintenance calls for brushing off the screen as needed. The configuration of the inlet screens and the required maintenance will protect the unit from oxygen starvation due to insufficient air flow. The blower is equipped with an inlet air filter. If this filter becomes blocked with debris it could cause oxygen starvation of the biomass. The blower inlet filter should be checked every 6 months and replaced as needed. The vent pipe could also become blocked, causing insufficient air flow out of the reactor. The vent screen should be checked for debris, and if the pipe is blocked, a drain augur can be used to clean out the line. If a vent pipe is used, vent holes in the pipe should be clean of debris. There is a possibility that the air line from the blower could become blocked. If this condition is suspected, disconnect the air line from the blower and check for blockage. A drain augur can be used to check the entire length of air line. If the drains in your house require an unusual amount of time to drain, the septic tank may require pumping out. Since there are no components underground which require repair or maintenance, there is no need to gain manual access to any underground components of the FAST system.

It is recommended that the installer keep the following minimum inventory of spare parts per the number of units sold. The quantities listed are minimums. If field experience suggests additional components or quantities are required, this list may be expanded.

QUANTITY	DESCRIPTION
2 units	Blower
2 units	Control Assembly
1-10 units	Air Filter

The FAST wastewater treatment system requires a constant supply of oxygen and food for the biomass. Should the blower stop, air flow through the aeration pipe will stop, cutting off the supply of oxygen to the biomass. A prolonged absence of oxygen will seriously affect the condition of the biomass. When the blower is operating, it will emit a humming sound. If the blower is not operating, first determine whether an electrical power outage has occurred in your community. The system is equipped with a light on the control panel and an alarm horn. Should the light illuminate or flash and the horn activate, check the breaker to ensure it has not tripped. If the breaker has tripped, attempt to reset it. The alarm horn may be shut off by pushing the silence button. Pushing the silence button will not reactivate the unit, only silence the horn.

Flood water may cover the septic tank unit, the blower housing, or both, if the FAST system is installed in a low-lying area. Electrical equipment located in flooded areas presents an electrical hazard. Stay out of a flooded area. Failure to do so may result in electrical shock causing death or serious bodily injury. Should water cover the blower housing, immediately disconnect electrical power to the blower at your house circuit breaker box by switching the switch to the off position, then call your service technician. Do not attempt to restore electrical power to the blower. The service technician must inspect and evaluate the condition of the FAST unit before electrical power is restored. Water-covering the septic tank unit can be tolerated if there is no backup in the system. Backup is characterized by wastewater flowing back into the house or slow movement of wastewater in the drains. Anyone coming in contact with wastewater must remove any contaminated clothing and thoroughly wash all exposed body areas with soap and water.

14.5.2 Observed Conditions

Operations and maintenance for this system were relatively uneventful for this unit with one exception. During the initial phase of sampling it was noted that the dissolved oxygen (DO) levels of effluent samples were low, but not of a level that prompted troubleshooting the system. During three successive site visits in August and September of 1998, field DO measurements revealed a substantial decrease to levels of 0.4 mg/L or less. Prior to that the lowest recorded effluent DO was 1.4 mg/L with the average being 2.8 mg/L. At that time the system was examined for the cause of this oxygen deficiency. The system blower was operating properly but most of the output was lost by substantial leakage around loose unglued PVC pipe joints. The sampler pushed the joints back together as securely as possible to enable aeration again. On the next sample visit the blower piping was glued and the connection from the pipe to blower was made with a flexible rubber coupling secured by worm drive hose clamps. For the remainder of the sampling period the field observed DO averaged 5.9 mg/L.

This experience with the FAST system reveals a deficiency in installation quality control. The air pipe installation was haphazard. Failure to glue PVC joints in a pressurized system is unacceptable. An analysis of the effluent DO levels during the course of the sampling period leads to the conclusion that this system was probably losing air since it was installed with the leakage gradually increasing over time. The highest effluent DO levels were consistently recorded after the pipe joints were glued on September 30, 1998. This system had been in place for less than two years when the air supply failed. This lead to an immediate degradation in performance. This type of failure is very difficult for the homeowner to discover. The blower appeared to be operating properly. The only alert came from the measured DO levels that decreased during sampling. The homeowner was conscientious regarding system maintenance. He seemed to be familiar with the system and performed blower maintenance as suggested by the manufacturer, cleaning the air inlet filter yearly.

System operation and maintenance experience on this system leads to the following recommendations for all systems that utilize aeration:

- Both blower and pipe must be secured in place.
- All PVC joints must be glued.
- Connection from the blower to the pipe should be by a PVC union, PVC compression coupling, or flexible rubber coupling secured by worm drive hose clamps.

- The homeowner should be provided with instructions and a check list from the manufacturer for simple system troubleshooting to determine operating status.

14.6 Reported System Performance

Testing of the Bio-Microbics, Inc., Single Home FAST Treatment Plant Model 23-001-750 was conducted under the provisions of NSF Standard 40 for Individual Aerobic Wastewater Treatment Plants (July 1990). NSF Standard 40 was developed by the NSF Joint Committee on Wastewater Technology. The performance evaluation was conducted at the NSF Wastewater Technology Test Facility in Chelsea, Michigan, using wastewater diverted from the Chelsea municipal wastewater collection system. The evaluation consisted of six months of testing, during which a seven week stress test was conducted. The evaluation consisted of three weeks of dosing without sampling to allow for plant start-up, sixteen weeks of dosing at design flow, seven weeks of stress test and five weeks of dosing at design flow. Sampling started in the fall and continued into spring, covering a full range of operating temperatures.

Standard 40, in Section H. (3) of Appendix A, provides for exclusion of up to ten percent of the effluent sample days, not to exceed one during stress testing, in completing the pass/fail determination. No sample days were excluded in this evaluation. Over the course of the evaluation, the average effluent BOD, was 9 mg/L , ranging between <5 mg/L and 24 mg/L , and the average effluent suspended solids was 7 mg/L, ranging between <5 mg/L and 27 mg/L. The pH ranged from 7.5 to 8.2. The maximum arithmetic mean of seven consecutive sample days was 14 mg for BOD, and l2 mg/L for suspended solids, both well below the allowed maximum of 45 mg/L. The maximum arithmetic mean of 30 consecutive sample days was l2 mg/L for BOD_5 and 8 mg/L for suspended solids, both well below the allowed maximum of 30 mg/L. The observed removal rates, which ranged from 92 to 95 percent for BOD5 and 95 to 97 percent for suspended solids, were consistently above the requirement of 85 percent. The effluent pH during the entire evaluation ranged between 7.5 and 8.2, within the required range of 6.0 to 9.0. The plant also met the requirements for noise levels (less than 60 dba at a distance of 20 feet) and color, threshold odor, oily film and foam. The Single Home FAST Treatment Plant Model 23-011-750 produced an effluent that successfully met the performance requirements established by NSF Standard 40 for Class I effluent (see Table 14.1).

Table 14.1 NSF Nitrogen Test Data for the FAST Unit.

Sample Date	Ammonia – N		Nitrate- N (Units are mg/L)		Total Kjeldahl - N	
	Influent	Effluent	Influent	Effluent	Influent	Effluent
9/17	22	3.3	<0.5	0.7	32	6.1
9/29	29	2.2	<0.5	4.1	36	4.7
9/24	34	4.0	<0.5	4.6	39	6.2
9/27	25	3.5	<0.5	3.9	33	5.4
10/1	28	4.7	<0.5	1.5	35	8.0
10/4	21	6.0	<0.5	1.4	31	9.4

Anderson, et. al., (1997) report on work done to evaluate the FAST system in the Florida Keys to minimize the impact of nutrients from onsite systems. Negative impacts to the Florida Keys coral reef ecology have been documented in recent years, and water quality degradation from nutrient overloading is a suspected cause. To protect the waters of the Florida Keys from further degradation), the Florida Keys National Marine Sanctuary (FKNMS) was established by the federal government in 1990. In 1993 the U.S. Environmental Protection Agency (EPA) identified domestic wastewater as a major source of nutrient loading to FKNMS waters. Many dwellings and commercial establishments in the Keys use onsite wastewater treatment systems (OWTS), so the Florida Keys Onsite Wastewater Nutrient Reduction Systems (OWNRS) Demonstration Project was initiated in 1995 to demonstrate the use of OWTS. It was hypothesized that these systems could significantly improve treatment and reduce the concentrations of nutrients discharged to the near-shore environment of the Keys.

A test facility was designed and constructed to evaluate various OWNRS simultaneously, under controlled conditions, using a common wastewater source. Five principal wastewater treatment process streams were operated concurrently at the test facility. Unit processes evaluated included attached growth and suspended growth biological processes (both aerobic and anaerobic), physical/chemical processes (adsorption, precipitation, ion exchange) and natural systems utilizing drip irrigation for plant uptake and evapotranspiration, influent and effluent quality were monitored monthly with 24-hour flow-composite samples. The principal treatment unit in process stream 3 is a proprietary unit known as the Bio-Microbics FAST^{tm} aerobic unit. This unit uses fixed-film activated sludge (FAS) treatment. The treatment is a combination suspended growth and attached growth aerobic biological process. This system provides nitrification of the effluent before discharge, and also provides denitrification by mixing activated sludge biomass through anaerobic zones. This mixing causes alternating aerobic and anaerobic conditions that favor the growth of denitrifying microorganisms and conversion of nitrate to nitrous oxide or nitrogen gas. Additional processes would be required for phosphorus removal following this system. Results after the first year of operation indicate that OWNRS effluent concentrations of 5 mg/L CBOD, 5 mg/L TSS, 10 mg/L total nitrogen, and 1 mg/L total phosphorus are achievable without chemical addition using combinations of the processes tested. Although excellent treatment was achieved with ONVNRS, significant additional construction, operation, and maintenance was required for these systems in comparison with conventional OWTS.

14.7 Field Trial Results

The FAST system was evaluated for 37 weeks from the April 29, 1998 through January 5, 1999.

14.7.1 Flow Characterization

A detailed flow characterization study was conducted for this site from July 24 through August 5 (Julian day 205-217) 1998. This study provided information about the flow patterns from household activities based on hour to hour and day to day variations. The system was monitored with a Campbell Scientific CR500 data acquisition system (DAS) for a 2 week period where all flow events were recorded. During the 13-day test the DAS monitored 314 discrete pump cycles. Some of these pump cycles were the result of an accumulation of more than one small flow event. Other pump cycles were part of large flow events that would occur during bathing and laundry activities. The overall frequency of flow events as indicated by the number of pump cycles is shown in Figure 14.3. This household exhibits a large variation in both hourly and daily usage rates. It is believed to be related to the makeup of the household, 2 working adults. This is apparent in the large areas of the chart where no mid-day use is recorded. The mean hydraulic hourly load profile for this site is illustrated in Figure 14.4. The relatively low standard deviation in hourly load profile indicates a consistent use pattern. The only significant variation noted occurred on the afternoon of August 2 when a large usage occurred. The exact nature of this usage is not known. In addition, this data indicates that our sampling program which used a composite of flows collected from 4:00 PM to about 9:00 AM captured

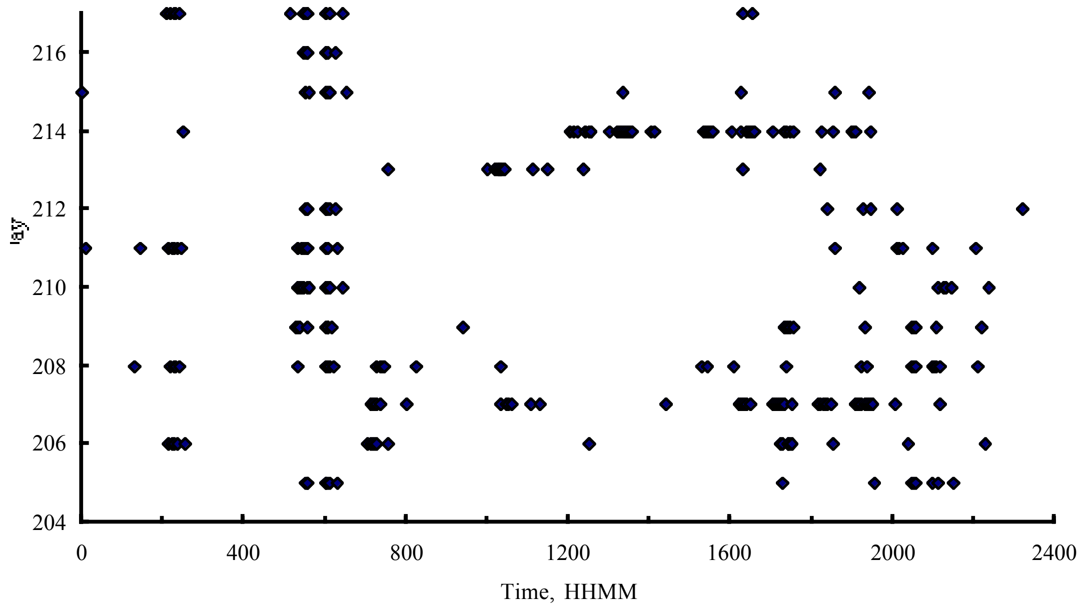

Figure 14.3. Frequency of Pump Events Recorded During the Flow Characterization Study.

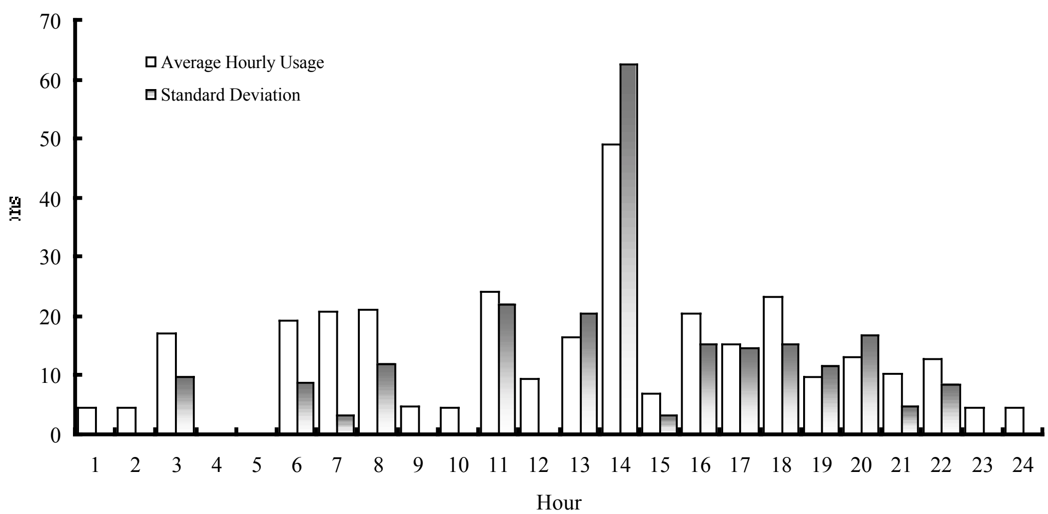

Figure 14.4. Mean and Standard Deviation of Daily Flows Recorded During the Flow Characterization Study.

about 75 percent of the flow based on time. Sampling did not collect a number of events that occurred during the mid-part of the day. However, with this household this was a relatively small amount. The time required to accomplish this collection could be incorporated in the study.

Mean flows determined for days of the week during the characterization study are shown in Figure 14.5. This data indicates that the mean flows varied from very small amounts (66 gallons) to large amounts (295 gallons). Large usage rates were observed on Sundays. With a 2 person working household this large weekend usage may be due to laundry flow events. The mean daily flow recorded for this period was 122 gallons per day.

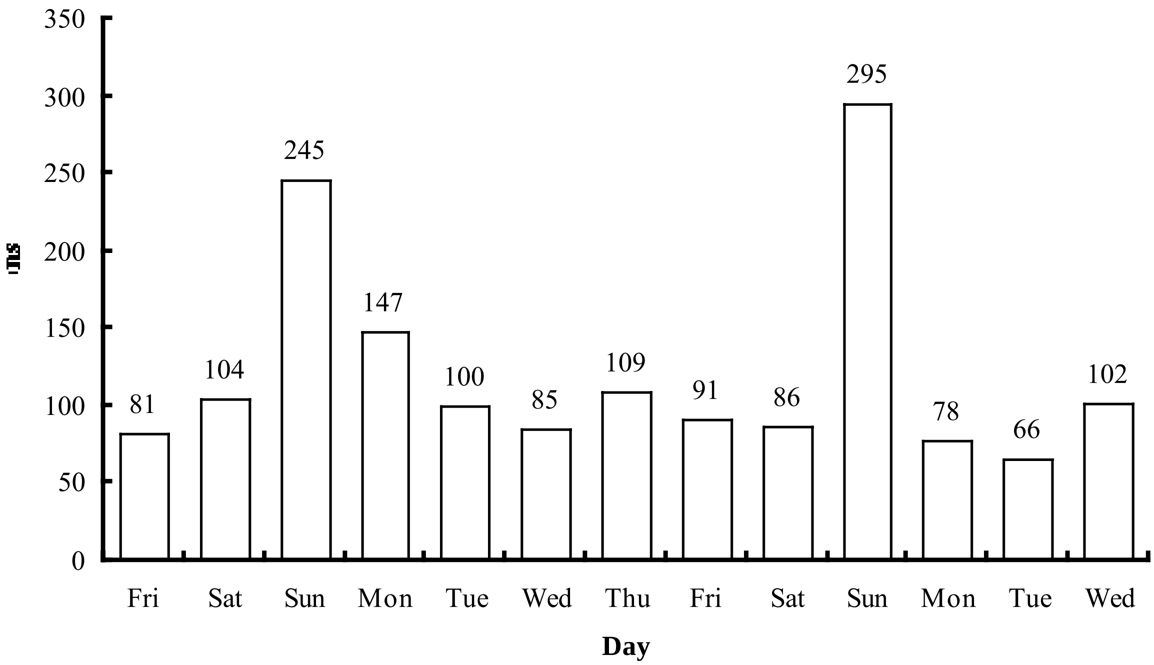

Figure 14.5. Mean Daily Flows for the Flow Characterization Study.

14.7.2 Hydraulic Analysis

The hydraulics of most reactor vessels fall somewhere in between complete mix flow (CMF) and plug flow (PF), which are the two ideal reactor models. This is termed non-ideal flow, which can be described by two basic models, the plug flow with dispersion model and the tanks in series model. The flow pattern for the FAST system was analyzed by determining the residence time distribution (RTD) of material flowing through the vessel. The method for the hydraulic analysis for the FAST system is given in the Methods (Chapter 8) section of this report.

A tracer study for this system was conducted from July 23 to August 6, 1998. On July 23, 1998, a total of 1,134.3 grams of bromide (Br) were introduced into the FAST system as a sodium bromide solution by the methods outlined in Chapter 8. During the tracer study, the concentration (C) of Br on the system effluent was measured and recorded as a function of elapsed time in hours from the beginning of the input of tracer into the system. This concentration vs. time (C vs. t) data was used directly and in conjunction with flow models to predict actual system behavior in terms of detention time and flow patterns. The C vs. t data for the FAST system was used to construct a C vs. t curve, which is shown in Figure 14.6.

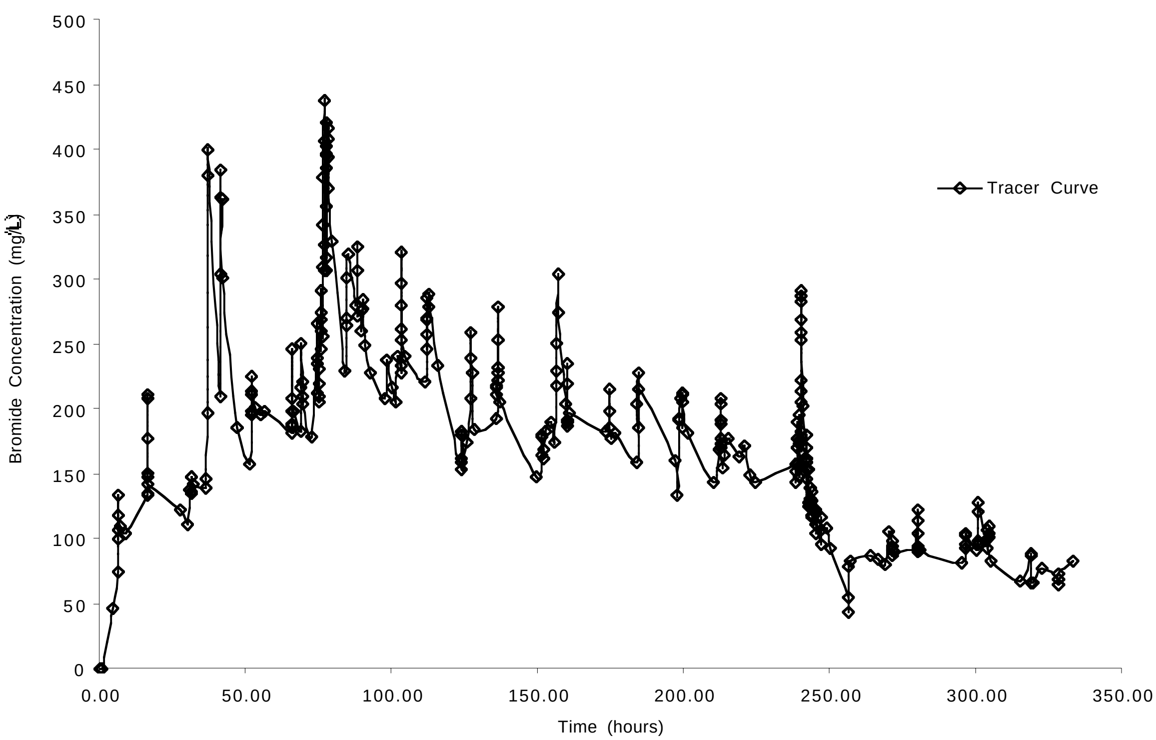

Figure 14.6. FAST System Tracer Study Concentration vs. Time Curve.

In addition, the overall shape of the C vs. t curve was used to look for evidence of short-circuiting in the reactor vessel. Based on the shape of the C vs. t curve for the FAST system, there was no evidence of short-circuiting in the system, which would have shown up as a sharp and early peak in the C vs. t curve.

As mentioned in Chapter 11, the mean and variance of a tracer curve are directly related to the system detention time and are the two quantities for describing tracer curves that are used in all areas of tracer experimentation. As previously mentioned in Chapter 8, the mean is the mean detention time of the system, while the variance tells how spread out in time the curve is. Using equation 8.3 (Chapter 8) the calculated mean detention time for the FAST system is as follows:

$$\text{Mean} = \sum(t_{i+1} + t_i)\,(C_{i+1} + C_I)\,(t_{i+1} - t_i)/\,2\sum(C_{i+1} + C_i)\,(t_{i+1} - t_i)$$
$$= 145.58 \text{ Hours}$$

The variance was calculated using equation 8.4 and is as follows:

$$\text{Variance} = \left(\sum(t_i + t_{i+1})^2\,(C_i + C_{i+1})\,(t_{i+1} - t_i)/\,4\sum(C_i + C_{i+1})\,(t_{i+1} - t_i)\right) - \text{mean}^2$$
$$= 7{,}011.05 \text{ hours}^2$$

The calculated mean, of 145.58 hours, along with the C vs. t data was used to construct the E curve for the system using the methods outlined by Levenspiel (1993) and which were discussed in general in Chapter 8. Namely, $E_t = C_i$/area, and $E_\theta = E_t$ *(mean) and θ = t/mean. The E curve for the Fast system is shown in Figure 14.7.

To determine which flow pattern approximated the FAST system, the E curve generated from measured C vs. t data was compared to theoretical E curves, such as the ones shown in Figure 11.8 of Chapter 11. From this comparison, it was determined that the FAST system E curve did not approximate the ideal mixed flow region or the ideal plug flow region. Therefore, the FAST E curve fell somewhere in between the two extremes of complete mix flow and plug flow, namely, the intermediate region that can be modeled by plug flow with dispersion or complete mix reactors in series.

As mentioned in Chapter 11, theoretical E curves (Levenspiel, 1993) like the ones shown on Figure 11.8, were developed for different vessel dispersion numbers (D/uL values). These theoretical E curves were constructed for the closed vessel situation by numerical methods using

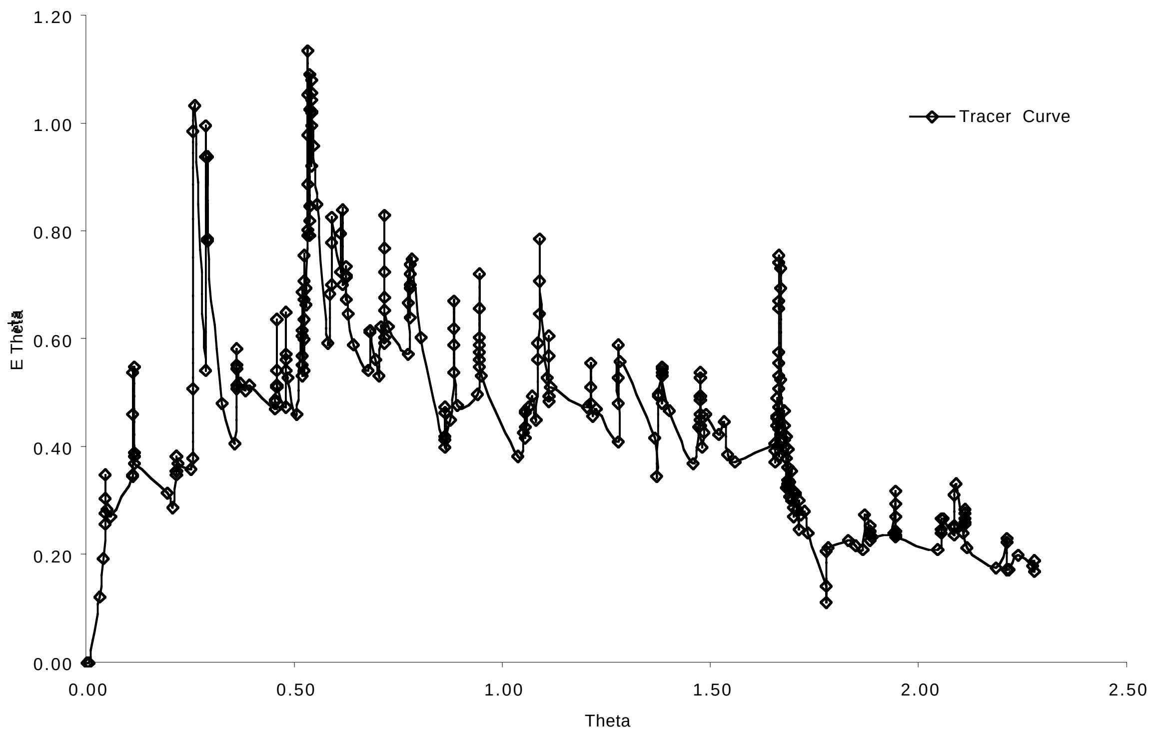

Figure 14.7. FAST System E (θ) Curve.

equation 11.2 in the previous Chapter. For the FAST system the assumption of closed vessel was also made since the tracer entered and left the system in small pipes relative to the total volume of the system.

The next step in the FAST system flow pattern analysis was to simulate the system using the Impulse program to determine how well the actual system approximated the plug flow with dispersion model. The Impulse program was used to simulate the FAST system as a plug flow reactor with dispersion. The Impulse program utilizes equation 11.2 (Chapter 11) to determine the theoretical best fit curve to the measured C vs. t data by running the program in the regression mode with regressable parameters or variables, such as inlet flowrates, reactor volume, dispersion number and inlet concentrations. The fit of the experimental data is based on equating the variances of the two curves about the center of gravity (mean residence time) of the distribution (Weber, 1972). The best fit of the simulated curve vs. the experimental (tracer) curve was determined by visual inspection from the plotted Impulse output of C vs. t data.

As discussed in Chapter 8, two scenarios, each for the plug flow with dispersion and complete mix flow models, were used to simulate the FAST system. The first scenario, using the plug flow with dispersion flow model, consisted of holding the reactor vessel volume constant and allowing Impulse to calculate the reactor influent flowrate and concentration, while insuring that the amount of bromide tracer remained at the actual amount of 1,134.3 grams. Figure 14.8 shows the tracer curve vs. the simulated curve using this scenario. The Impulse program also calculated the dispersion number (D/uL) as 1.49. While the curve fit is excellent, the program output of calculated variables did not match the actual system. For example, the calculated dispersion number using equation 11.2 by a trial and error procedure was as follows:

$$\text{Variance/mean}^2 = 7{,}011.05/(145.58)^2 = 0.331$$

$$\text{Variance/mean}^2 = 0.331 = 2\ D/uL - 2(D/uL)^2\ (1 - e^{-uL/D})$$

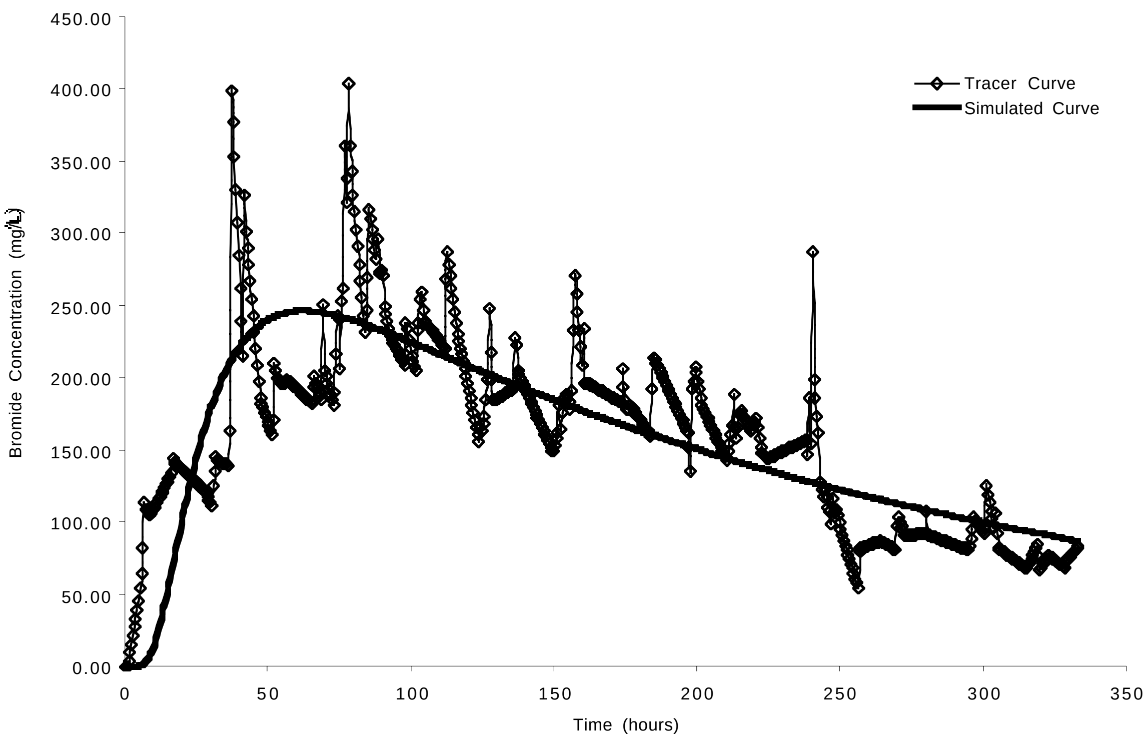

Figure 14.8. Tracer Curve vs. Plug Flow w/Dispersion Simulated Curve for the FAST System - Calculated D/uL = 1.49.

By trial and error D/uL = 0.21, which is within the plug flow with dispersion range. However, the Impulse calculated dispersion number of 1.49 is highly skeptical since for values of D/uL>1 the assumption of plug flow with dispersion should not be used (Levenspiel, 1993).

Also, the calculated inlet flowrate of 7.72 liters/hour is considerably less than the actual average flowrate of 19.11 liters/hour (121.2 gallons/day). Furthermore, a reactor volume of 2,100 (555 gallons) was used vs. the actual reactor volume of 3,028 liters (800 gallons) in order to maintain the mass of bromide tracer at the actual value of 1,134.3 grams.

The second modeling scenario, using the plug flow with dispersion flow model, was carried out by holding the inlet flowrate and concentration constant, with the flowrate at 19.11 liters/hour (121.2 gallons/day), which was the actual calculated average flowrate and the concentration necessary to insure a pulse input of 1,134.3 grams of bromide tracer into the system. The reactor vessel volume was varied by Impulse. Figure 14.9 shows a plot of the simulated curve vs. the tracer curve constructed by using the output of C vs. t data from Impulse. Although, the curve fit seems worse than the first scenario, the dispersion number (D/uL)

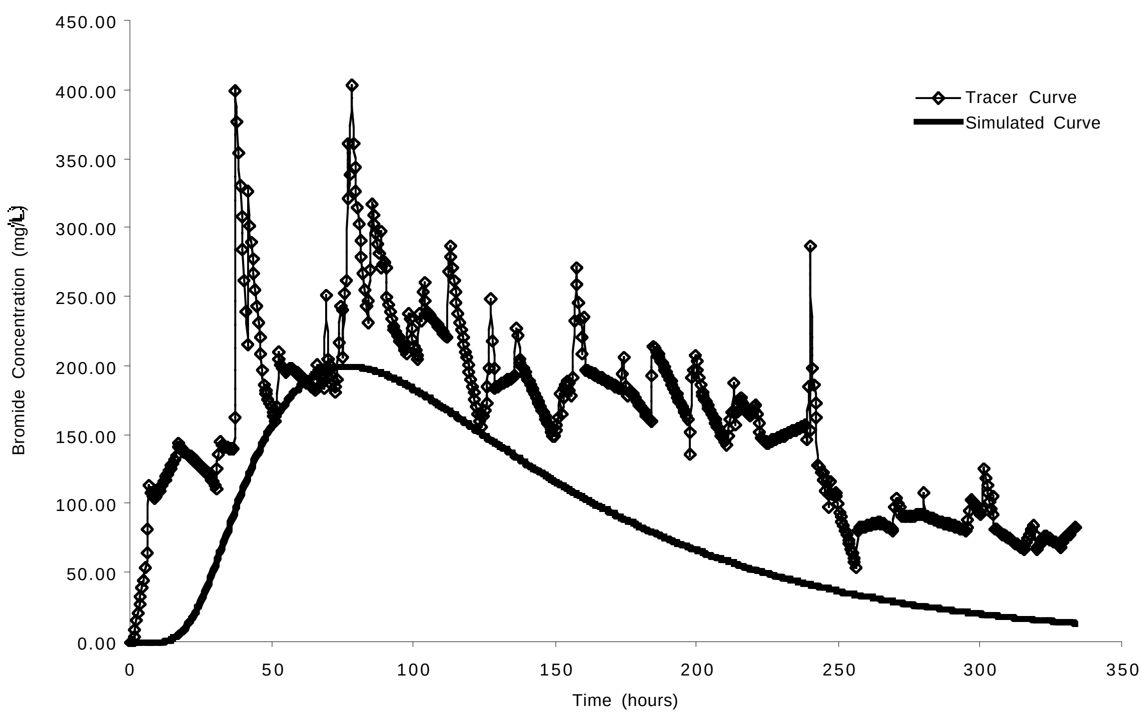

Figure 14.9. Tracer Curve vs. Plug Flow w/Dispersion Simulated Curve for the FAST System - Calculated D/uL = 0.29.

calculated by Impulse for this scenario was 0.29, which is comparable to the calculated value of 0.21 using the actual C vs. t tracer data. Furthermore, the influent flowrate is the actual average flowrate already calculated as 19.11 liters/hour (121.2 gallons/day). The reactor vessel volume calculated by Impulse was 2,624 liters (693 gallons), which is comparable to the actual reactor vessel volume of 3,028 liters (800 gallons).

As with the Whitewater system, the only concern with the second plug flow with dispersion scenario is, that the simulated curve seems to have a good fit to the experimental curve at the beginning of the experiment but seems to diverge towards the end of the experiment. Again, it is speculated at this point that this may have been caused by experimental error in calculating the bromide concentration. The Cole Palmer Bromide Electrode model 27502-04 seemed to show a shift in calibration curve as a function of time elapsed from the beginning of the experiment. This shift was found to be attributed to a deposition of a certain type of material on the probe during the course of the two week experiment. This may have caused the tail-end of the tracer curve to not approach zero toward the end of the two-week experiment, as is demonstrated by the Impulse simulated curve (Figure 14.9).

Based on the results of the Impulse simulation of the FAST system as a plug flow with dispersion reactor, the dispersion numbers (D/uL) of 0.21 (C vs. t experimental data), and the value of 0.29 calculated by Impulse, show that the deviation from plug flow is large (D/uL>0.01). Therefore, as D/uL approaches infinity, the system approaches complete mixed flow behavior. While the FAST system exhibited Plug Flow with dispersion behavior, a simulation as a complete mix flow reactor was performed using Impulse for illustration purposes. Once again, two different scenarios were used for the complete mixed flow simulations: 1) vary inlet flowrate, and 2) vary reactor vessel volume. Figures 14.10 (vary flowrate) and 14.11 (vary reactor vessel volume) show the tracer curve vs. the simulated curves for the complete mix flow simulations for the two scenarios.

The inputs to the Impulse program for the simulation of the FAST system as complete mix flow for the two scenarios were: 1) a constant reactor vessel volume of 2600 liters (687 gallons) to satisfy the actual pulse tracer input of 1,134.3 grams, and 2) constant flowrate of 19.11 liters/hour (121.2 gallons/day) and constant inlet concentration to satisfy the pulse tracer input of 1,134.3 grams. The Impulse program output for the scenario where the inlet flowrate

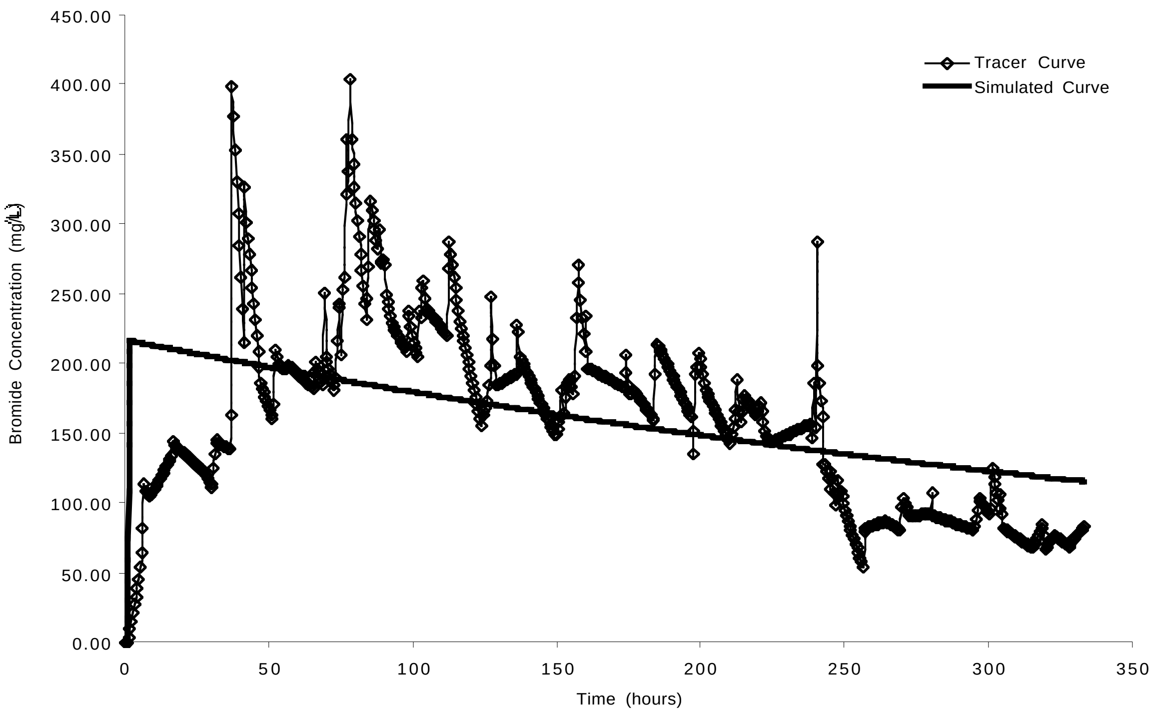

Figure 14.10. Tracer Curve vs. Mixed Flow Simulated Curve for FAST System - Vary Inlet Flowrate.

was varied was the C vs. t data used to construct the simulated curve shown in Figure 14.10, and an inlet flowrate of 4.9 liters/hour (31.1 gallons/day). The Impulse program output for the scenario where the reactor vessel volume was varied was the C vs. t data used to construct the simulated curve shown in Figure 14.11, and a reactor vessel volume of 2976.4 liters (786.4 gallons). Once more the scenario where the reactor vessel volume was varied demonstrated a worse curve fit than the scenario where the inlet flowrate was varied. As with the plug flow with dispersion simulations, the greater deviation from the tracer curve for the second scenario (vary reactor vessel volume) is suspected to have been caused by the experimental error in measuring the bromide concentration.

In conclusion, the observed curve fit and comparison of calculated flowrates and volumes shows that the FAST system exhibited plug flow with dispersion, but approximates the complete mix flow extreme rather than the plug flow extreme, which is typical of these type of systems (aerated systems). Aeration provides good mixing, however, not enough to allow the system to fall into the complete mix flow regime. Also, the FAST system consisted of two chambers, a

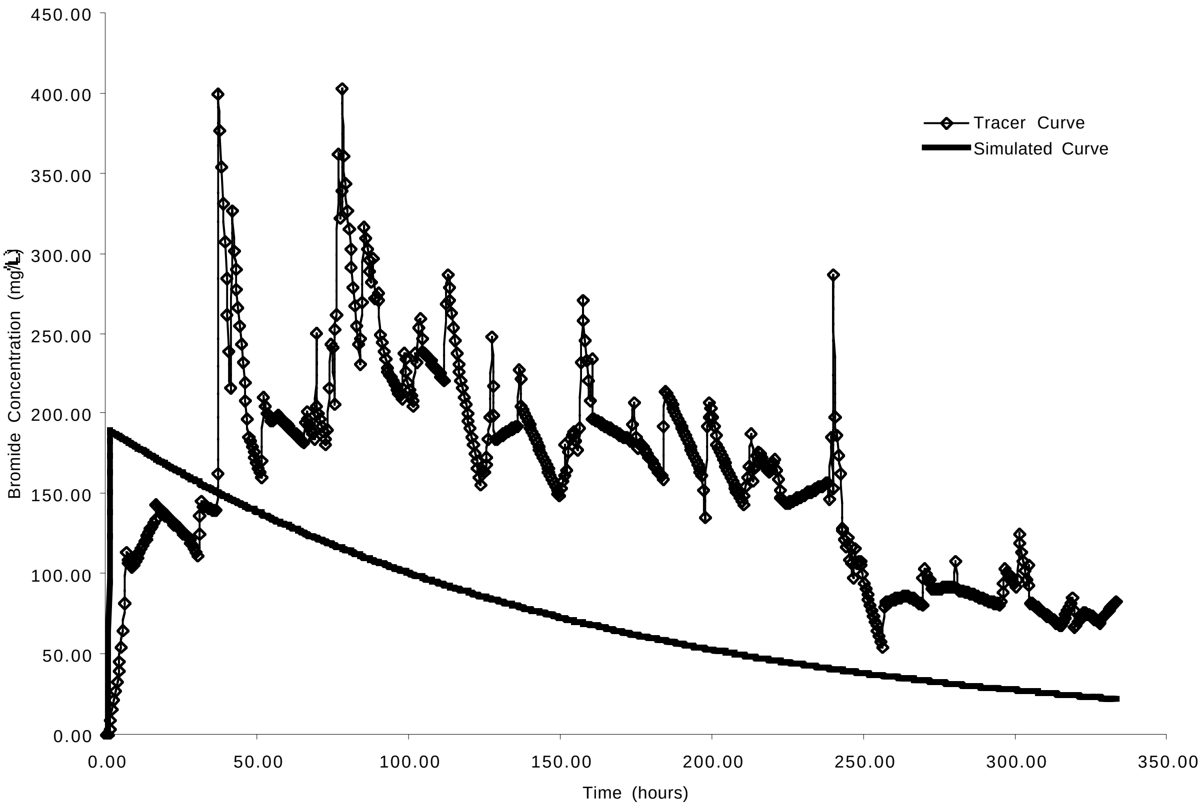

Figure 14.11. Tracer Curve vs. Mixed Flow Simulated Curve for FAST System - Vary Reactor Vessel Volume.

settling chamber and an aerated chamber as described previously. Therefore, in the settling chamber there is less mixing than in the second chamber, thus preventing the system from exhibiting the complete mix flow regime. A summary of the Impulse simulation results as well as the values obtained from the actual tracer data are shown in Table 14.2.

14.7.3 Water Quality Analysis

The FAST system was performance tested for 37 weeks from the April 29, 1998 through January 5, 1999 with 19 influent and effluent samples collected. The average daily flow recorded by the onsite flow meter indicated as shown in Figure 14.12, that flow varied between 78 and 171 gpd over this period with an overall average flow of 119.6 gpd. This averaged flow

Table 14.2 Summary of Impulse Simulation Results for FAST System.

Scenario	Flow in (gpd)	Mass of Br Added (g)	Reactor Vol. (gallons)	Dispersion # D/uL	Curve Fit
Actual/Tracer	121.2	1,134.3	800	0.21	N/A
PF w/Disp. (vary flow)	48.95	1,134.3	555	1.49	Excellent
PF w/Disp. (vary volume)	121.2	1,134.3	693	0.29	Fair
CMF (vary flow)	31.1	1,134.3	687	N/A	Fair
CMF (vary volume)	121.2	1,134.3	786	N/A	Poor

over the test period was consistent and steady and showed no major differing patterns. The flow was recorded on the effluent side of the treatment unit and thus some damping of extreme flow events could be expected as the treatment unit acted to equalize flow. Table 14.3 summarizes different flow conditions, other system conditions calculated from flow as well as detention times.

Table 14.3 Summary of Different Flow Conditions and Unit Operation Detention Times for the Test System.

Parameter	Design Flow	Mean Flow	Flow Study	Tracer Study
Flow, gpd	450	161.8	134.3	147.5
Total Unit Volume, gal	909	909	909	909
System Detention Time, days	1.81	5.61	6.76	6.14
% Difference from Design	0	67.8	73.2	70.6
Reactor Detention Time, days	1.43	4.45	5.36	4.88
Clarifier Detention Time, days	0.38	1.17	1.41	1.28
Clarifier Overflow Rate, gpd/ft^2	29.20	9.45	7.84	8.61

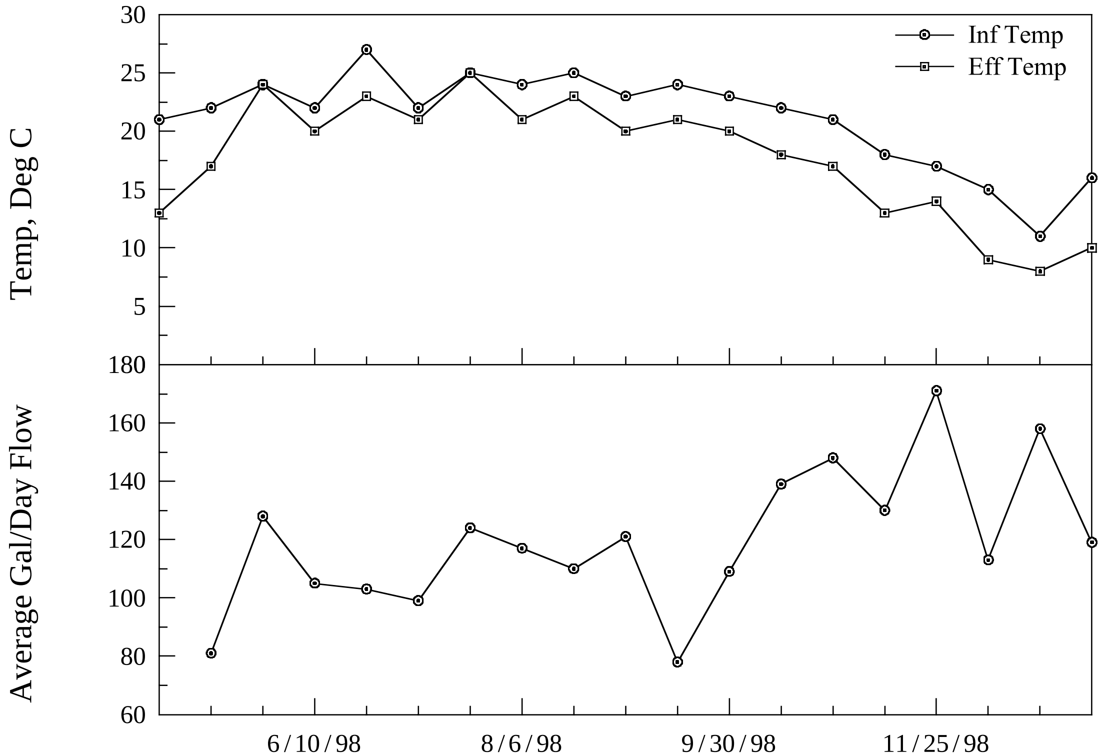

Figure 14.12. Co-FAST System Experimental Data.

Temperature data (Figure 14.12) for the site indicated no significant difference between the influent and effluent with mean values of 21.2 and 17.7 °C, respectively. Effluent temperature, reflecting the actual operating temperature of the process, varied from 8 to 25 °C over the study period. These temperature differences can affect the performance of biological treatment process particularly nitrification, which is very ineffective at temperatures less than 15 °C. The temperature issue is critical for this system. Looking at the DO data, there is reason to believe the factory installers may have hooked up the air system wrong. This was discovered when the air quit entirely in mid-September. This means there were only 4 weeks of performance with both high D0 and high temperature. The electrical conductivity (EC) reflects the total dissolved solids in a particular water sample. In many cases the change in EC can be an indicator of evaporative processes or the addition of chemicals such as from a water softener, laundry operations, reverse osmosis unit, electroplating, or photo developing processes. Many of these processes may add chemicals that cannot be detected by other measurement techniques or require very specialized and expensive analysis. The EC for influent and effluent samples for the FAST test site are shown in Figure 14.13. The influent and effluent did not vary significantly with mean values of 1,702 and 1,801 respectively. Typically, water softeners are piped to the waste disposal unit and the plumbing code requires that this method be followed for all installations. The EC concentrations encountered did not indicate any unusual activities at this household. No large spikes in the data were noted. Large increases or shock loads can detrimentally affect many biological waste processes.

Chloride data (Figure 14.13) for the influent and effluent averaged 1,096 and 882 mg/L respectively. Chloride can be directly contributed by a water softener, but no large spikes in concentration in the influent were observed. Sulfate data (Figure 14.13), exhibited very little variability in the influent. The influent and effluent sulfate concentrations averaged 39.36 and 60.17 mg/L, respectively and were not significantly different. Dissolved or ortho-phosphorus concentrations in the influent and effluent from this test system (Figure 14.13) averaged 10.1 and 3.26 mg/L, respectively. The influent and effluent concentrations were significantly different ($p = 0.00000144$) with a system percent removal of ortho-phosphorus of 67 percent.

The data for pH is shown in Figure 14.14. Maintaining a near neutral pH (6 to 8) is important for the stability of biological processes. Many cleaners and drain openers and other chemicals can drastically raise or lower pH and impact system performance. Influent pH values ranged from 5.78 to 7.96 over the course of the study while effluent values ranged from 7.67 to 8.77. The mean influent and effluent pH values were 7.11 and 8.09, respectively and were not significantly different. While some extreme values were encountered in the influent, the effluent appeared to be much less variable and certainly within range to maintain good biological treatment.

The TSS data values for the system shown in Figure 14.14 indicated an influent event that elevated TSS concentrations to 5,990 mg/L. The average influent concentrations were determined to be 1,356 mg/L with a standard deviation of 1,441. The effluent values averaged 28.33 mg/L with a standard deviation of 23.61. The influent and effluent were significantly

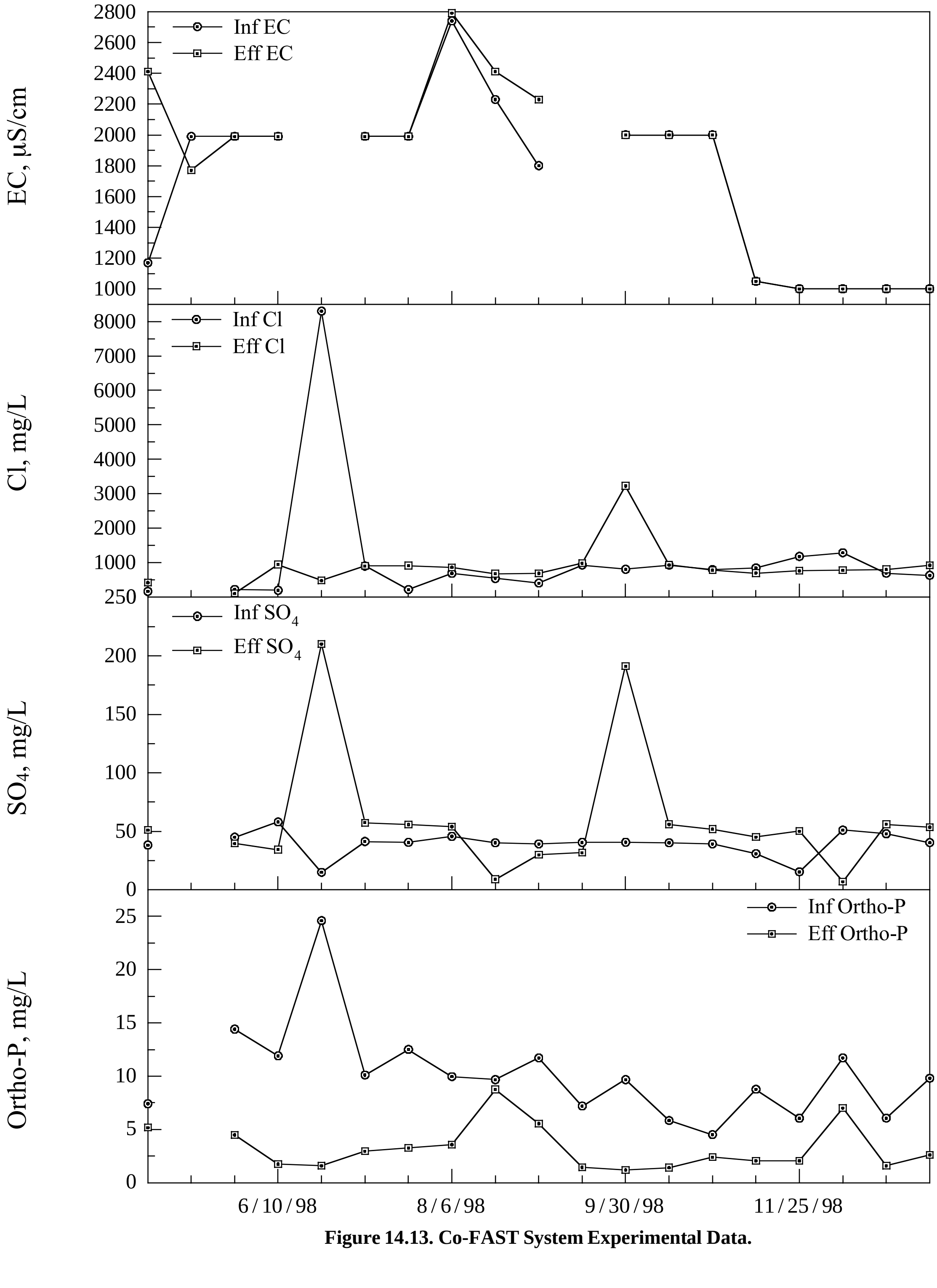

Figure 14.13. Co-FAST System Experimental Data.

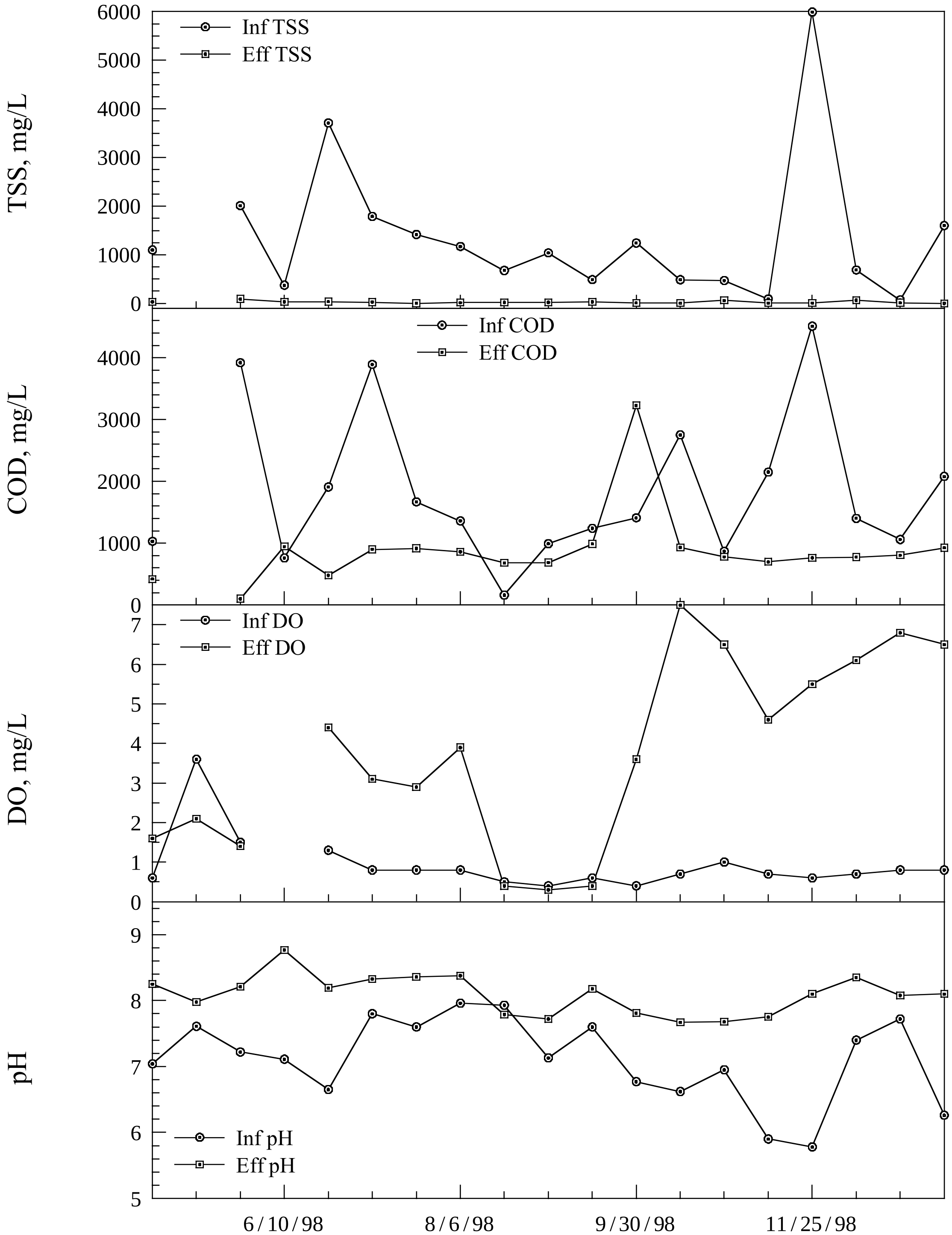

Figure 14.14. Co-FAST System Experimental Data.

different at a *p* value = 0.00042. The calculated percent removal TSS was 98 percent for this system. This indicated excellent removal of TSS consistently below 30 mg/L, but not below 20mg/L or less.

The BOD_5 values for this system were not graphed in part because only half the values were provided by the laboratory after samples had been submitted. The BOD_5 values for the influent has a mean of 523 mg/L. The BOD_5 values for the effluent had a mean of 76 mg/L. The influent and effluent were significantly different at a *p* value = 0.00617.

The COD data values for the system shown in Figure 14.14 indicated some influent events that elevated COD concentrations over 4,500 mg/L. The average influent concentrations were determined to be 1,842 mg/L with a standard deviation of 1,202. The effluent values averaged 882 mg/L with a standard deviation of 488.2. The influent and effluent were significantly different at a *p* value = 0.001275. The calculated percent removal of COD was 52 percent for this system.

The DO data values for the system, shown in Figure 14.14, indicated some variation in influent DO with concentration ranging form below 0.4 mg/L to 3.6 mg/L The average influent concentrations were determined to be 0.92 mg/L with a standard deviation of 0.72. The effluent values averaged 3.76 mg/L with a standard deviation of 2.38. The influent and effluent were significantly different at a *p* value = 0.000029. The average reported value measured in the system effluent during operation of the air supply system should be acceptable for aerobic treatment (2 to 4 mg/L). However, there are some interesting changes in the effluent DO concentration from the beginning of the data set to the end. Notice that from 4/29/98 to 8/6/98 the DO concentrations were in the range of 1.4 – 4.4. This level of DO is normally considered sufficient to prevent oxygen from being a limiting resource in a suspended growth system. The aeration in the reactor was vigorous enough during this period to allow a person on the surface to hear the mixing. It is worthy to note that since this is a fixed film system and this DO concentration is from the top of the fixed film media, the 2 mg/L in the effluent may be too low as a target for acceptable performance. During sampling on 8/19/98 it was noted that the effluent DO was down to 0.4 mg/L and it stayed depressed until 9/30/98. Apparently the system installers had not glued the pressure piping from the blower to the reactor. This pipe came apart between 8/6/98 and 8/19/98. On 9/30/98 the pipe was uncovered and reattached and the following sampling event the pipe was glued by the sampling team. The DO levels after this repair ranged from 3.6 to 7.5 with an average value of 5.9 mg/L compared to an effluent value of 2.8 in the first part of the data set. It appears that the air was leaking out during the first period of the study. It also appears from the nitrate data that this system is still oxygen limited at an average effluent DO of 2.8 mg/L. Note that once the air supply was repaired it took more than 2 weeks for the DO levels to come up to steady state. It appears that the system had built up a waste load in the treatment unit when the air was off. It required some time for the microbes to consume the excess food.

The ammonia data values for the system, shown in Figure 14.15, indicated wide variation influent ammonia with concentration ranging from 5 mg/L to 89.2 mg/L The average influent concentrations were determined to be 49.21 mg/L with a standard deviation of 21.19. The effluent values averaged 15.97 mg/L with a standard deviation of 17.75. The influent and effluent were significantly different at a p value = 0.0000127. There are some interesting things shown in this ammonia data. The reduction in ammonia is not unexpected in light of the DO concentrations, about 4.3 mg oxygen are required to convert each mg of ammonia nitrate. From 4/20/98 to 6/24/98, there was very little ammonia oxidation. From 7/9/98 to 8/6/98, the microbes were oxidizing 97% of the ammonia. From 8/19/98 to 9/18/98 the ammonia oxidation efficiency dropped dramatically. This reduction in ammonia oxidation corresponds to the low DO period. Immediately following the repair of the air supply system, the nitrification resumed and was nearly complete. The microbial population, once established, recovered from the Oxygen deprivation in less than 2 weeks without apparent harm.

The nitrate data values for the system, shown in Figure 14.15, indicated almost no variation in the influent nitrate concentration. The average influent concentrations were determined to be 0.05 mg/L with a standard deviation of 0.0. The effluent values averaged 7.0 mg/L with a standard deviation of 10.56. The influent and effluent were significantly different at a p value = 0.00436. Again, a closer look at this data provides some insight into the system. During the period from 4/20/98 to 6/24/98, when there is no measured ammonia oxidation, the nitrates are below detect. During the period 7/9/98 to 8/6/98 when there was significant ammonia oxidation, there was significant nitrate measured in 2 out of 3 samples. The levels increased from 0.5 mg/L to 10.7 mg/L. During the air system failure (8/19/98 to 9/18/98), there were no nitrates formed.

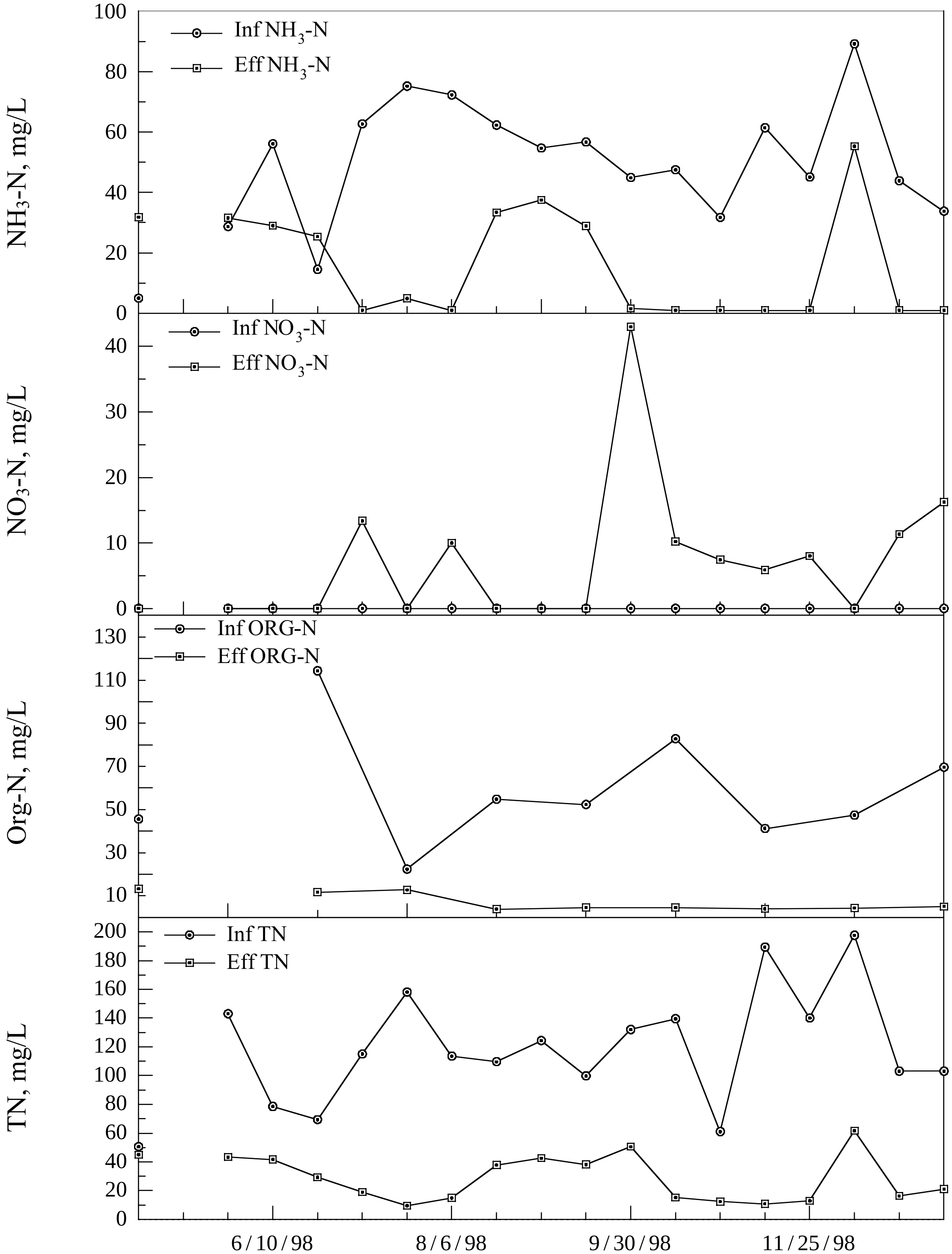

Figure 14.15. Co-FAST System Experimental Data.

As soon as the oxygen supply system was repaired, there was once again high nitrate in the effluent (8.8 mg/L). Note the nitrate point immediately following the repair of the air supply system. The nitrate level jumped to 42 mg/L, which corresponds to a period of depressed DO levels.

The organic-n data values for the system, shown in Figure 14.15, indicated some variation in influent organic-n with concentrations ranging from 36 mg/L to over 108.8 mg/L. The average influent concentrations was determined to be 70.43 mg/L with a standard deviation of 26.0. The effluent values averaged 14.23 mg/L with a standard deviation of 16.78. On a long term average basis, this corresponds to an 80% removal of organic nitrogen.

The total nitrogen data values for the system, shown in Figure 14.15, indicated some variation in influent total nitrogen with concentrations ranging form 22.4 mg/L to 127.9 mg/L. The average influent concentration was determined to be 68.96 mg/L with a standard deviation of 30.26. The effluent values averaged 6.10 mg/L with a standard deviation of 3.25. There is a significant reduction in total nitrogen through the system, a comparison of the influent and effluent mean values yields a p value = $1.18E^{-10}$. Note that the removal efficiencies for TN closely mimic the trends shown on the ammonia and nitrate plots. The overall removal efficiency during periods of effective nitrification was 83%. During periods of ineffective nitrification the TN removal efficiency dropped to 63%. This implies that there is a significant level of nitrification/denitrification occurring.

The fecal coliform (FC) data for the FAST system was highly variable with mean influent and effluent values of 6.02×10^5 and 2.04×10^4 cfu/100mL, respectively. The data is shown in Table D-1 in the Appendix. If the lowest value is excluded, this resulted in a removal rate of 99.0 percent, which is a 2 log removal. Influent values ranged from 6.0×10^4 to 1.98×10^6 and effluent ranged from 200 to 7.8×10^4cfu/100mL. This level of fecal destruction is not a concern under current regulations. However, if future regulations allow variances based on treatment performance, the fecal destruction may be of interest. In the proposed Wisconsin regulation of 1999, a standard minimum separation distance between the limiting soil layer and the bottom of the drainfield is set at three feet. This separation distance can be reduced based on “treatment credits”, which are performance based and independent of the treatment unit. Fecals $>10^4$ require the full separation distance. Fecals between 10^3 and 10^4, earn sufficient treatment credits to reduce the separation distance to 2 feet, and if fecals are $<10^3$ the separation distance is reduced to 1 foot. These extractions are a simplification of the actual Wisconsin Code, but indicate the relative importance of fecal removal in another state's regulations. New Mexico does not currently use fecals to grant variances of any kind.

14.8 Conclusions

The FAST system was performance tested for 37 weeks from the April 29, 1998 through January 5, 1999 with 19 influent and effluent samples collected. The flow characterization and reactor tracer analysis, were also performed on the system. In addition, the installation, maintenance, and operation of the system were evaluated. The system as tested evaluated performance based on effluent from the house as the influent to the system. The septic tank is considered part of the FAST system.

The hydraulic analysis of the system indicated no short circuiting or unusual flow problems with the system. The system exhibited a flow pattern that was closely related to a plug flow with dispersion system approximating the mixed flow extreme. The average measured flow, over the study period was 119.6 gpd, which compared favorably to the flow measured during the tracer studies and flow characterization study. These flows were significantly less than the design flow (500 gpd), for the unit.

A summary of operating parameters for this system is shown in Table 14.4. This data indicates that the system as tested was operating well during significant portions of the testing.

Table 14.4 Operating and Design System Parameters for FAST System.

Parameter	Loading lbs/day	Effluent mg/L	Percent Removal %
BOD_5	0.52	76.0	85.5
COD	1.84	882.4	52.1
TSS	1.35	28.3	97.9
NH_3-N	0.05	16.0	67.5
TN	0.12	29.0	75.5
Ortho-P	0.01	3.3	67.3

The critical regulatory parameters for BOD_5, TSS, TN, and FC are shown in Table 14.5 for the test system. These data indicated that average values generated for the system were well above the recommended performance standards for any of the zones listed. However the study period covered startup for the system and data collected from this period skewed the performance of the system. The best data for the field trials are shown in Table 14.5, along with the long term averages and the proposed target levels.

Table 14.5 Comparison of FAST Data and Proposed Performance Standards.

	Field Trial Data		Performance Standards		
Parameter	Overall Mean	Best Results[1]	Zone A	Zone B	Zone C
BOD_5, mg/L	76.0	60	30	20	15
TSS, mg/L	28.3	8.8	30	20	15
TN, mg/L	29.0	13.3	30	20	10
FC, cfu/100mL	2.04×10^4	200	100	50	1
COD, mg/L	882.4	593	N/A	N/A	N/A
Ortho-P, mg/L	3.3	1.6	N/A	N/A	N/A

[1] Mean of lowest five data points (throwing out the lowest value) with recorded flow event

Observations regarding system operation and maintenance indicated that the manufacturer-supplied information was well documented and was transferred to the installer and the homeowner. However, the degree of training appeared inadequate to insure proper operation of the system. Maintenance problems were encountered with the aeration system on several occasions and for extended periods during the testing period. There was no alarm provided to indicate low aeration pressure in the event of a piping disconnect. The inspection of the air supply system is critical for this type of system.

When the air supply system was functional, this system did a reasonably good job of TN reduction. This is an "off the shelf" commercial package, and it is not clear that the manufacturer has optimized the system for removal of nitrogen. It appears that increasing the recycle ratio may improve the denitrification. There will clearly be an optimum point, and too much recycle will stop denitrification completely. It is also possible that denitrification could be drastically improved by recycling to the first compartment of the septic tank, which is not only anoxic, but is also rich in carbon. There is a strong possibility that future units of this type will perform significantly better than this unit. The manufacturers are just now becoming aware of the market associated with denitrification.

Chapter 15 - Septic System

15.1 Site Description

The system tested was a standard septic tank. This system is characterized as an anaerobic solid/liquid separation system. The treatment system is designed to treat 500 gallons per day of domestic wastewater. A local manufacturer provided the unit. An approved local onsite system installer installed the unit. The test site was a three-bedroom house with a detached art studio located in Bernalillo County east of the Sandia Mountains in a fractured bedrock region. The home has a water softener for at least part of the household water. Because the installation was in an area with shallow fractured bedrock, it was necessary to excavate the rock and back fill with an appropriate leachfield material. The exact size and location of the leachfield are unknown. The system layout with sample sumps and required setbacks is shown in Figure 15.1. A side view of the septic tank is shown in Figure 15.2. Sampling was set up using methods outlined in the previous methods section.

15.2 Process Description

The septic tank is the most widely used onsite wastewater treatment option in the United States. Currently, about 25% of the new homes being constructed in this country use septic tanks for treatment prior to disposal of home wastewater. Septic tanks for single-family homes are usually purchased "off the shelf," ready for installation, and are normally designed in accordance with local codes. Most local codes are based on ASTM Standards such as the "Specification for Precast Concrete Septic Tanks" (C1227-96) and other standards dealing with waterproofing and or specifications for pipes and fittings, etc.

Septic tanks are buried, water tight receptacles designed and constructed to receive wastewater from a home, to separate solids from the liquid, to provide limited digestion of organic matter, to store solids, and to allow the clarified liquid to discharge for further treatment and disposal. The settleable solids and partially decomposed sludge settle to the bottom of the tank and accumulate. A scum of lightweight material (including fats and greases) rises to the top. The septic tank should remove nearly all settleable solids and floatable grease and scum so that a reasonably clear liquid is discharged into the soil absorption field. This allows the field to

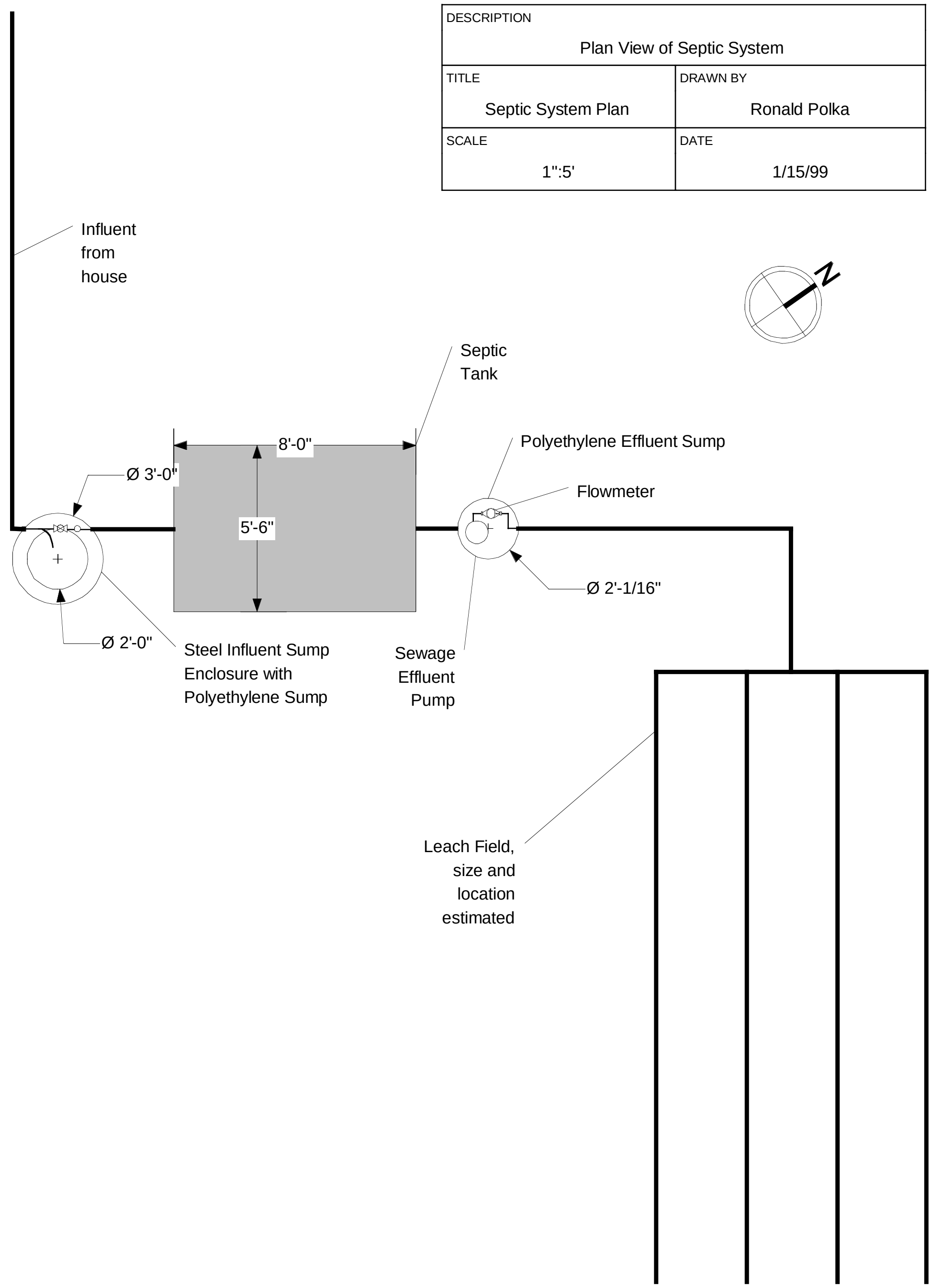

Figure 15.1. Septic System Plan View.

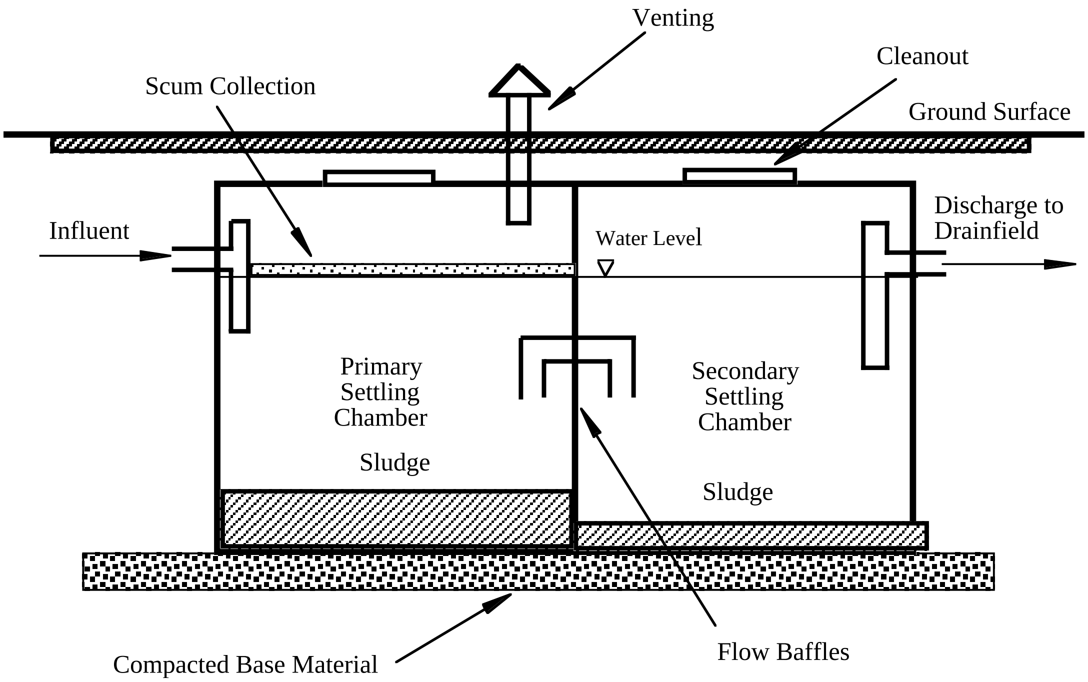

Figure 15.2. Side View of the Septic Tank Onsite Wastewater Treatment System.

operate more efficiently. The partially clarified liquid is allowed to flow through an outlet structure just below the floating scum layer. Proper use of baffles, tees, and ells protects against scum outflow. The soil absorption field filters and treats the clarified septic tank effluent. Removing the solids from the wastewater protects the soil absorption system from clogging and premature failure. In addition to removing solids, the septic tank also permits digestion of a portion of the solids and stores the undigested portion. Up to 50 percent of the solids retained in the tank decompose. The remaining solids accumulate in the tank. To accomplish this, the tank must provide:

- Liquid volume sufficient for a 24-hr fluid retention time at maximum sludge depth and scum accumulation (Weibel, et. al., 1954).

- Inlet and outlet devices to prevent the discharge of sludge or scum in the effluent.

- Sufficient sludge storage space to prevent the discharge of sludge or scum in the effluent.

- Venting provisions to allow for the escape of accumulated methane and hydrogen sulfide gases.

Septic tanks are normally the first component of an onsite system. They must be followed by polishing treatment and/or disposal units. In most instances, septic tank effluent is discharged to a soil absorption field where the wastewater percolates down through the soil, and is further treated by the soil bacteria. Clarified liquid can be disposed of to a soil absorption system, a soil mound, a lagoon or another disposal system. Septic tanks may also be used as pretreatment for an advanced onsite wastewater treatment system.

A septic tank is a combined clarifier and anaerobic treatment unit. It is reasonable to expect 40-60% removal of solids by sedimentation. The solids represent 20-30% of the BOD in the influent wastewater. This will be removed with the solids. If the tank detention time exceeds three days, it is reasonable to expect up to 50% removal of soluble BOD due to anaerobic degradation. Since the system is anaerobic, no nitrification will occur. Without nitrification, very little denitrification will occur.

The first step in selecting a tank volume is to determine the average volume of wastewater produced per day. Ideally, this is done by metering wastewater flows for a given period, but that is seldom feasible, particularly if a septic tank system is for a building still under construction. Design capacity of most septic tanks is based on the number of bedrooms per home and the average number of persons per bedroom. For example, assume the average wastewater contribution is about 45 gpcd (170 Lpcd) (Small Scale Management Project, 1978). A safety factor, can be assumed and a value of 75 gpcd (284 Lpcd) can be combined with a potential maximum dwelling density of two persons per bedroom, yielding a theoretical design flow of 150 gal/bedroom/day (570 L/bedroom/day). A theoretical tank volume of 2 to 3 times the design daily flow is common, resulting in a total tank design capacity of 300 to 450 gal per bedroom. While not ideal, this method allows the designer to assign required septic tank capacities based solely on the number of bedrooms (Table 15.1) (Otis, et.al., 1980). Unfortunately, hourly and daily flows from the home can vary greatly. During high flow periods, higher solids concentrations are discharged from the septic tank. Well-designed, two compartment tanks reduce the effect of peak hour loads.

Use of garbage grinders increases both the settleable and floatable solids in the wastewater and their accumulation rates in the septic tank. U.S. Public Health Service (USPHS) studies indicate that the increase in the sludge and scum accumulation rate is about 37%. This means either more frequent pumping or a larger tank to keep the pumping frequency down. It is

Table 15.1 Septic Tank Volume Requirements.

Housing	Federal Health Authority	U.S. Public Plumbing Service	Uniform Range of State Code	Requirements
Min., gal	750	750	750	500-1,000
1-2 bedrooms	750	750	750	500-1,000
3 bedrooms	900	900	1,000	900-1,500
4 bedrooms	1,000	1,000	1,200	1,000-2,000
5 bedrooms	1,250	1,250	1,500	1,100-2,000
Additional bdrm (ea)	250	150	150	

generally a good idea to avoid the use of garbage grinders with onsite systems. A common expedient is to add 250 gal (946) to the tank size when garbage grinders are used, although this volume is arbitrary. Some guidelines recommend adding 50% to the recommended size because of the extra load to the system.

Another key factor in the design and performance of septic tanks is the relationship between surface area, surge storage, discharge rate, and outlet exit velocity. These parameters affect the hydraulic efficiency and sludge retention capacity of the tank. Tanks with greater surface area and shallower depth are preferred, because increased liquid surface area increases surge storage capacity; a given inflowing volume creates a smaller rise in water depth, a slower discharge rate, and a lower exit velocity. The damping of flow surges due to increased surface area allow a longer time for separation of sludge and scum that are mixed by turbulence resulting from the influent surge (Weibel, et al., 1954).

The balance between storage of sludge and suppression of surges leads to the following guidelines on dimensions. The depth from the invert of the outlet to the floor of the tank, (liquid depth) of any septic tank or compartment thereof, shall not be less than four feet (1.2 m). A liquid depth greater than six feet six inches (2 m) shall not be considered in determining tank capacity. The length of a septic tank should not be less than five feet (1.5 m) and should be approximately two to three times the width; but no tank or compartment thereof shall have an inside horizontal dimension less than two feet (0.6 m). The minimum inside diameter of a vertical cylindrical septic tank shall be five feet (1.5 m).

The inlet turbulence in a single-compartment tank causes mixing of the sludge with the wastewater in the clear space. The increased velocity of the water in the vertical leg of the outlet tee re-suspends previously captured solids. The rising gases produced by anaerobic digestion interfere with particle settling and re-suspend previously captured solids, which then are lost in the effluent. Therefore, the inlet to a septic tank should be designed to dissipate the energy of the incoming water, to minimize turbulence, and to prevent short-circuiting. The inlet should preferably be either a sanitary tee or baffle. The baffler should be small enough so that it is flushed out each time the house sewer flows, yet keeps floating solids from blocking the inlet. The invert radius in a tee helps dissipate energy in the transition from horizontal to vertical flow, and prevents dripping that, at the proper frequency, can amplify water surface oscillations and increase inter-compartmental mixing. The vertical leg of the inlet tee should extend below the liquid surface. This minimizes induced turbulence by dissipating as much energy in the inlet as possible.

The outlet structure's ability to retain sludge and scum is a major factor in overall performance. The outlet of a septic tank can be a tee, a baffle, or some special structure. The outlet must have the proper submergence and height above liquid level such that the sludge and scum clear spaces balance, and proper venting of sludge gases is provided. The 10 States Standard of Practice (Great Lakes Upper Mississippi River Board of State Sanitary Engineers, 1980), recommends putting the outlet baffle, or sanitary tee, no more than 6" from the tank outlet and suggests extending it to within 1" of the tank top and to a depth of at least 40% of the liquid depth. Although the Manual of Septic Tank Practices recommends the 40% submergence for the baffle, other studies have shown that shallower submergence decreases solids discharges and allows for greater sludge accumulation, and thus for less frequent pumping (Weibel, et.al., 1954). Reducing the exit velocity, reduces the opportunity for solids and scum to escape through the outlet. This can be accomplished by using a 6-in. (15-cm) outlet riser instead of a 4-in. (10-cm) outlet riser will reduce the exit velocity from 0.025 ft/sec to 0.011 ft/sec (0.76 cm/sec to 0.34 cm/sec) a reduction of 56% (Weibel, et.al., 1954). The minimum requirements for the inlet and outlet structures as set by Ten States Standards are reviewed in the following paragraphs.

The inlet connection to the septic tank shall not be less than four inches (10 cm) inside diameter and enter the tank at least three inches (7.6 cm) above the liquid level. The inlet connection of the septic tank and each compartment thereof shall be submerged by means of a vented tee or baffled so as to obtain effective retention of scum and sludge. The inlet tee or baffle shall extend above the liquid level to a point not less than one inch (2.5 cm) from the underside of the top of the tank to assure system venting. The inlet baffle or tee shall extend below the liquid level at least six inches (15 cm), but not more than 20 percent of the total liquid depth. Baffles shall be located a minimum of six inches (15 cm) from the inlet pipe.

The outlet connection to the septic tank shall not be less than four inches (10 cm). The outlet connection of the septic tank and each compartment shall be submerged by means of a vented tee or baffled so as to obtain effective retention of scum and sludge. The outlet tee or baffle shall extend above the liquid level not less than 20 percent of the liquid depth in tanks with straight vertical sides and 15 percent of the liquid depth in horizontal cylindrical tanks. The outlet tee or baffle shall extend below the liquid level a distance equal to 40 percent of the liquid depth for tanks with straight vertical sides and 35 percent of the liquid depth in horizontal cylindrical tanks. Baffles shall be located no more than six inches (15 cm) from the outlet pipe. There shall be at least a one inch (2.5 cm) vent space between the underside of the top of the tank and the top of the outlet baffle or tee.

A septic tank shall provide an air space having a volume not less than ten percent of the liquid capacity of the tank. Clearance of at least nine inches (23 cm) shall be provided between the maximum liquid level at the outlet and the highest point on the ceiling of the tank body.

Adequate access to each compartment of the tank for inspection and sludge removal shall be provided by a manhole or removable cover with a minimum dimension of twenty inches (51 cm). Manholes are usually placed over both the inlet and the outlet to permit cleaning behind the baffles. The manhole cover should extend above the actual septic tank to a depth not more than 6 in. (15 cm) below the finished grade. The actual cover can extend to the ground surface if a proper seal is provided to prevent the escape of odors and accidental entry into the tank. In addition, small inspection pipes can be placed over the inlet and outlet to allow inspection without having to remove the manhole. When tanks have been installed with the top of the tank more than twelve inches (30 cm), below grade, an inspection pipe of not less than six inches (15 cm), diameter extending through the cover to a point, above the tank not more than six inches (15 cm), below finished ground level shall be provided. The point at which the inspection pipe passes through the cover shall be so located that a downward projection of the pipe clears the inlet and outlet device by not less than two inches (5 cm). The top of the inspection pipe shall be provided with a readily removable watertight cap or plug and its location shall be marked at the ground surface. The inlet and outlet devices shall be made accessible by removable covers, manholes, or by properly located inspection ports as already described.

Recent trends in septic tank design favor multiple, rather than single, compartmented tanks. When a tank is properly divided into compartments, BOD and SS removal are improved. The benefits of compartmentation are due largely to hydraulic isolation, and to the reduction or elimination of inter-compartmental mixing. Mixing can occur by two means: water oscillation and true turbulence. Oscillatory mixing can be minimized by making compartments unequal in size (commonly the second compartment is 1/3 to 1/2 the size of the first), reducing flow-through area, and using an ell to connect compartments (Jones, 1978). Minimum size of the first compartment shall be 500 gallons (1,890 L). The minimum plan dimension of any compartment of the septic tank shall be twenty-four inches (61 cm). In the first compartment, some mixing of sludge and scum with the liquid always occurs due to induced turbulence from entering wastewater and the digestive process. The second compartment receives the clarified effluent from the first compartment. Most of the time it receives this hydraulic load at a lower rate and with less turbulence than does the first compartment, and, thus, better conditions exist for settling low density solids. These conditions lead to longer working periods before the pump-out of solids is necessary which produces an improvement in overall performance.

A vent space shall be provided between compartments of a septic tank. Inlets and outlets to a compartmented tank shall be proportioned and located as for a single tank. The liquid connection between compartments shall consist of two or more openings equally spaced across the width of the tank with an area equal to three times the inlet and located at a depth of 40 percent of the liquid depth as measured from the liquid level.

Materials

Septic tanks are constructed of the following materials unless dictated by state or local regulations:

- Pre-cast reinforced concrete.
- Fiberglass.
- Cast-in-place reinforced concrete.
- Concrete blocks (cinder blocks)with concrete-filled cores. Insert reinforcing rods in, the cores before filling them. Waterproof the tank inside and out.

- Asphalt-coated steel (useful life is 2 to 5 yr.).
- Polyethylene.
- Other materials meeting state and local strength and durability criteria.

The most commonly used construction material for septic tanks is concrete. Most individual home septic tanks are pre-cast reinforced concrete because they are easy to install and readily available. The walls have a thickness of 3 to 4 in. (8 to 10 cm), and the tank is sealed for water tightness after installation with two coats of bituminous coating. Care must be taken to seal around the inlet and discharge pipes with a bonding compound that will adhere both to concrete and to the inlet and outlet pipe. Cinder block construction, though not encouraged, is also common. The raw materials are inexpensive and available. It is recommended that the tank walls be waterproofed before the cover is cast. Steel is another type of material that has been used for septic tanks. The steel must be treated so as to be able to resist corrosion and decay. Such protection includes bituminous coating or other corrosion-resistant treatment. However, despite a corrosion-resistant coating, tanks deteriorate at the liquid level. Past history indicates that steel tanks have a short operational life (less than 10 years) due to corrosion (Weibel, et.al., 1949). Other materials include polyethylene and fiberglass. Plastic and fiberglass tanks are light, easily transported, and resistant to corrosion and decay. While these tanks have not had a good history, some manufacturers are now producing an excellent tank with increased strength. This minimizes the chance of damage during installation or when heavy machinery moves over it after burial.

Multi-Home and Commercial Wastewater

In some instances, a septic tank can serve several homes, or a commercial/institutional user such as a school, store, laundry, or restaurant. Whereas septic tanks for single-family homes must handle highly variable flows (i.e., approximately 45% of the total household flow occurs in the peak four hours), commercial systems must also be able to treat continuous wastewater flows for 8-16 hours a day as well as peak loadings. In addition, commercial wastewaters may present special problems that need to be handled prior to discharge to the septic tank (i.e., grease removal for restaurant wastewaters, and lint removal for laundry wastewater).

Multiple-compartment tanks for commercial /institutional flows should have the same design features as single-family home tanks discussed previously. These include: compartments separated by walls with ports or slits at proper elevations, proper venting, access to all compartments, and proper inlet and outlet design and submergence. Tanks with more than two compartments are not used frequently. The effect of a multiple-compartment tank can be accomplished by using two or more tanks in a series. A better construction arrangement, particularly for medium or large installations, is to connect special tank sections together into a unit having single end-walls and two compartments.

Larger tanks for commercial/institutional flows or for clusters of homes must be sized for the intended flow. Whenever possible with existing facilities, the flow should be metered to obtain accurate readings on average daily flows and flow peaks. For housing clusters, if the total flow cannot be measured, the individually metered or estimated flows (based on the expected population and the generation rate of 45 gal/cap/ day (170/cap/day) from each house must be summed to determine the design flow. For commercial /institutional applications, the flow estimate must be performed on a case by case basis. For flows between 750 and 1,500 gal/day (2,840 to 5,680 Lpd), the capacity of the tank is normally equal to 1- 1.5 days of wastewater flow. For flows between 1,500 and 15,000 gpd (5,680 to 56,800 Lpd), the minimum effective tank capacity can be calculated at 1,125 gal (4,260 L) plus 75% of the daily flow; or

$$V = 1{,}125 + 0.75Q \qquad (15.1)$$

where:

V = net volume of the tank (gal); and

Q = daily wastewater flow (gal)

For restaurants, size the septic tank at least twice as large as the values generated using the equations. Put tanks in series for cooling and coagulation of fats and greases. If garbage grinders are used, additional volume or extra sludge storage may be desired to minimize the frequency of pumping (Manual of Septic Tank Practices, 1967).

Dosing Tanks (Pumping Pits)

Where dosing tanks are provided, they shall meet the construction requirements specified for septic tanks. To provide storage, the total volume of the dosing tank should be equivalent to the average daily flow. The required volume of the dosing tank shall not be considered as any portion of the required volume of the septic tank. Dosing tanks must be provided with access ports sufficiently large to maintain the tank and pumps, and must be vented. In case of pump failure, a high water alarm switch set above the design volume of the dose, shall be provided on a separate circuit from the pump, and have an audible or visible alarm in the house. Pumps and control devices within the dosing tank shall be of an explosion proof design.

15.3 Unit Installation

A number of minimum separation distances (set backs) have been developed for protecting water supplies and homes from septic tank disposal systems. Septic tank location with respect to the home is usually determined by the slope of the land and the major bathroom and kitchen plumbing. The septic tanks should be at least 10' from the foundation wall and straight out from the point where the sewer pipe goes through the wall. Check local codes for distance requirements between septic tanks, foundation walls, and property lines. Locate the septic tank out of high vehicle traffic areas, since excessive loads can damage the tank. Avoid areas subject to flooding or discharge from downspouts. The tank should be located within 40' of an access road so a tank truck can clean the tank and avoid driving over the disposal field. Consider future home improvements such as sidewalks, patios, garages, and storage buildings when selecting the tank site. When a waste system needs maintenance or when ownership changes, an accurate sketch of the waste system is important. **NOTE**: Make a plan showing:

- Septic tank size and location.
- Openings for pumping out sludge.
- Soil treatment area size and location.
- Direction and distance of the septic tank and treatment area from the house.
- Installer’s name and address.

File the sketch in a place where it is readily available.

The most important requirement of installation is that the tank be placed on a level grade and at a depth that provides adequate gravity flow from the home and matches the invert elevation of the house sewer. The tank should be placed on undisturbed soil so that settling does not occur. If the excavation is dug too deep, it should be backfilled to the proper elevation with sand to provide an adequate bedding for the tank. Level the tank, then backfill immediately or fill the tank with water to prevent flotation or shifting. Tank performance can be impaired if a level position is not maintained, because inlet and outlet structures will not function properly.

Septic tank depth depends on house plumbing and whether gravity flow from a basement sewer drain is provided. A sewage treatment system works better and is more easily maintained if the soil treatment area is near the ground surface. Gravity flow throughout the system is desirable. Use a 4" plastic or rigid sewer pipe with watertight joints for the outlet pipe. Slope it at least 1% (1/8"/ft). Avoid running the outlet pipe under driveways to reduce the risk of crushing and freezing. If the outlet pipe does run under a driveway, protect the pipe from heavy loads by placing 6" of gravel around it.

Provide a riser from the access port of the septic tank and pumping chamber. Grade the area away from the riser to divert surface water. Make the riser opening at least 24" in the smallest dimension and use a heavy cover to prevent children from removing it. In deep installations, check the strength of the tank with your supplier. The tank must support the extra weight of the riser. Other considerations include:

- Cast iron inlet and outlet structures should be used in disturbed soil areas where tank settling may occur.
- Flotation collars should be used in areas with high groundwater potential.
- The tank should be placed so that the manhole is slightly below grade to prevent accidental entry.
- The tank should be placed in an area with easy access to alleviate pump-out problems.
- During installation, any damage to the watertight coating should be repaired. After installation, the tank should be tested for water tightness by filling with water.
- Care should be taken with installation in areas with large rocks to prevent undue localized stresses.
- Baffles, tees, and elbows should be made of durable and corrosion-proof materials. Fiberglass or acid-resistant concrete baffle materials are most suitable. Vitrified clay tile, plastic, and cast iron are best for tees and ells.
- To start septic action in a new tank, use it. The natural processes usually begin as the tank fills. It is not necessary to seed the system with commercial products.

15.3.1 Manufacturer's Recommendation

One of the major advantages of the septic tank is that it has no moving parts and, therefore, needs very little routine maintenance. A well designed and maintained concrete, fiberglass, or plastic tank should last for 50 years. Because of corrosion problems, steel tanks can be expected to last no more than 10 years. One cause of septic tank problems involves a failure to pump out the sludge solids when required. As the sludge depth increases, the effective liquid volume and detention time decrease. As this occurs, solids scouring increases, treatment efficiency falls off, and more solids escape through the outlet. No treatment process is capable of continuous operation without experiencing some type of residuals buildup. Obviously, improper operation and maintenance will impair performance. The best designed and operated septic tank/disposal field eventually fails unless sludge is periodically removed from the septic tank. Inadequate maintenance can cause sewage to back up into the house and solids to overflow to the soil disposal area. Flushing problem wastes (paper towels, bones, fats, diapers, etc.) into the system can clog piping.

Removal and disposal of these residuals is a very important and often neglected part of overall system O&M. The only way to prevent this is by periodic pumping of the tank. The frequency of pumping depends on several factors:

- Capacity of septic tank.
- Flow of wastewater (related to size of household).
- Volume of solids in wastewater (more solids if garbage disposal is used).

Table 15.2 (Mancl, 1998a) gives the estimated pumping frequencies according to septic tank capacity and household size. The frequencies were calculated to provide a minimum of 24 hours of wastewater retention assuming 50 percent digestion of the retained solids. For example, assume a 1500-gallon septic tank is used for a home with three bedrooms. If six people reside in a three-bedroom house, the tank should be pumped every 2.6 years. If the same system serves a family of two, the tank would be ready for pumping every 9.1 years. Older systems with smaller septic tanks may need to be pumped more often than once a year. More frequent pumping is needed if a garbage disposal is used. Following septic tank cleaning, all interior surfaces of the tank should be inspected for leaks and cracks using a strong light. Do not enter the tank or place your head in the tank during the visual inspection, but perform the inspection using a mirror. It is important to note that the soil absorption field will not fail immediately when a full tank is not

Table 15.2 Estimated Septic Tank Pumping Frequencies in Years
(For year-round residence).

Tank Size (gal) — **Household Size (Number of People)**

	1	2	3	4	5	6	7	8	9	10
500	5.8	2.6	1.5	1.0	0.7	0.4	0.3	0.2	0.1	-
750	9.1	4.2	2.6	1.8	1.3	1.0	0.7	0.6	0.4	0.3
1000	12.4	5.9	3.7	2.6	2.0	1.5	1.2	1.0	0.8	0.7
1250	15.6	7.5	4.8	3.4	2.6	2.0	1.7	1.4	1.2	1.0
1500	18.9	9.1	5.9	4.2	3.3	2.6	2.1	1.8	1.5	1.3
1750	22.1	10.7	6.9	5.0	3.9	3.1	2.6	2.2	1.9	1.6
2000	25.4	12.4	8.0	5.9	4.5	3.7	3.1	2.6	2.2	2.0
2250	28.6	14.0	9.1	6.7	5.2	4.2	3.5	3.0	2.6	2.3
2500	31.9	15.6	10.2	7.5	5.9	4.8	4.0	4.0	3.0	2.6

pumped. However, the septic tank is no longer protecting the soil absorption field from solids. Continued neglect will result in failure and the soil absorption field may need to be replaced. In some cases, replacement of the absorption area may not be possible due to site limitations.

Tanks should be inspected at intervals of 1-2 years to determine the rates of scum and sludge accumulation. If inspection programs are not carried out, a pump-out frequency of once every 3 to 5 years is reasonable. Once the characteristic sludge accumulation rate is known, inspection frequency can be adjusted accordingly. The manhole, not the inspection pipe, should be used for pumping so as to minimize the risk of harm to the inlet and outlet baffles. The inlet and outlet structures and key joints should be inspected for damage after each tank pump-out.

At least once a year, the depth of sludge and scum in the septic tank should be measured. When as a result of such measurement, the top of the sludge layer in the tank or any compartment of the tank is found to be less than twelve inches (30.5 cm) below the bottom of the outlet baffle or submerged pipe, or if the bottom of the scum layer is within three inches (7.6 cm) of the septic tank outlet baffle or submerged pipe, the tank shall be pumped and sanitary disposal made of the contents. Annual pumping may be substituted for measurement.

Scum can be measured with a stick to which a weighted flap has been hinged, or with any device that can be used to feel the bottom of the scum mat. The stick is forced through the mat, the hinged flap falls into a horizontal position, and the stick is raised until resistance from the bottom of the scum is felt. With the same tool, the distance to the bottom of the outlet device can be determined. A long stick wrapped with rough, white toweling and lowered to the bottom of the tank will show the depth of sludge and the liquid depth of the tank. The stick should be lowered behind the outlet device to avoid scum particles. After several minutes, the sludge layer can be distinguished by sludge particles clinging to the toweling. Other methods for measuring sludge include connecting a small pump to a clear plastic line and lowering the line until the pump starts to draw high solids concentrations.

SafetyNote: Pumped-out septic tanks often contain toxic gases; therefore, only an experienced person should attempt to enter or repair a septic tank if this should become necessary. The average homeowner should not enter a septic tank. Call a professional if tank maintenance is required. Septic tank gases are dangerous. Climbing into septic tanks can be very dangerous, as the tanks are full of toxic gases. When using the manhole, take every precaution possible, i.e., do not lower an individual into the tank without a proper air supply, and safety rope tied around chest or waist. Never go into a septic tank to retrieve someone who fell in and was overcome by toxic gases or the lack of oxygen without a self-contained breathing apparatus (SCBA). If a SCBA is not available the best thing to do is call for emergency services and put a fan at the top of the tank to blow in fresh air. Methane, an explosive, and hydrogen sulfide, a poison, are the major hazardous gases released during pumping. Torches or other flames near the septic tank opening can cause an explosion. Never lean into or enter a septic tank, even to save someone else, without proper breathing equipment. You could be poisoned or asphyxiated. When working on a tank, make sure it is well ventilated and someone is standing nearby.

Cleaning the Tank

At least once a year, dosing tanks and distribution boxes should be opened and settled solids removed as necessary. Septic tank pump and haul contractors can clean your tank. It is a good idea to supervise cleaning to ensure that it is done properly. To extract all the material from the tank, the scum layer must be broken up and the sludge layers stirred up into a liquid portion of the tank. This is usually done by alternately siphoning liquid from the tank and reinjecting it into the bottom of the tank. The septic tank should be pumped out through the large manhole, not the baffle inspection ports. Pumping out a tank through the baffle inspection ports can damage the baffles.

Before closing the tank, check the condition of the baffles. If they are missing or deteriorated, replace them with sanitary tees. It should never be necessary to enter a septic tank. Any work to replace the baffles or repair the tank should be made from the outside. The septic tank produces toxic gases that can kill a person in a matter of minutes. To facilitate future cleaning and inspection, install risers from the central manhole and inspection ports to the surface or near the surface before burying the tank. Also mark the location of the tank so that it can be easily found. Leaving solids in the septic tank to aid in starting the system is not necessary. When pumped, the septic tank must not be disinfected, washed, or scrubbed. Special chemicals are not needed to start activity in a septic tank.Special additives are not needed to improve or assist tank operation once it is under way. No chemical additives are needed to “clean” septic tanks. Such compounds may cause sludge bulking and decreased sludge digestion. However, ordinary amounts of bleaches, lyes, caustics, soaps, detergents, and drain cleaners do not harm the system. Other preparations, some of which claim to eliminate the need for septic tank pumping, are not necessary for proper operation and are of questionable value. Materials not readily decomposed (e.g., sanitary napkins, coffee grounds, cooking fats, bones, wet-strength towels, disposable diapers, facial tissues, cigarette butts) should never be flushed into a septic tank. They will not degrade in the tank, and can clog inlets, outlets, and the disposal systems.

No matter what the cause, septic system failure is a nuisance and a health hazard that should be corrected promptly. Failures can result in pollution of groundwater and associated wells. Some of the more common reasons for septic system failure are discussed here. These failures can be attributed to several causes. A trained sanitarian should diagnose the problem and make recommendations for corrective action. Leakage from septic tanks is often considered a minor factor; however, if tank leakage causes the level of the scum layer to drop below the outlet baffle, excessive scum discharges can occur. In the extreme case, the sludge layer will dry and compact, and normal tank cleaning practices will not remove it (Jones, 1978). If the tank is not watertight infiltration of standing groundwater into the tank can cause overloading of the tank and subsequent treatment and disposal components.

Using Too Much Water

Using more water than the soil can absorb is the most common reason for failure. The sewage is forced to the surface or backs up into the house. This problem is often the result of a change in water use habits, such as an increase in the size of the family or the addition of a water-using appliance. Surface water draining from roofs, driveways and roads onto the soil absorption field area can also put an extra load on the system. If the soil is saturated with water, even seasonally, it cannot accept any more water. The untreated wastewater will either surface or back-up.

Physical Damage

Driving, paving or building on top of a septic system can damage the soil absorption field. Pipes can shift or be crushed and the soil is compacted. Damage of this sort makes it difficult to locate the septic tank and prevents access for regular pumping. Tree roots can also clog the soil absorption field. Plant the area in grass, not trees or shrubs.

Improper Design and Construction

Improperly designed and constructed septic systems are doomed from the start. These systems usually fail in a few months because they are inadequately sized, installed in impermeable soils or not properly constructed. Four feet of unsaturated soil must be present beneath the soil absorption system to a limiting layer. Seasonal high groundwater, bedrock or impervious soil are all considered limiting layers. The soil beneath the drain field is the most important part of the septic system and must be properly evaluated and protected. If the soil layer is too thin, the wastewater will not be treated before it enters the groundwater. If the soil is too tight, it will not absorb all the wastewater, forcing it to the surface. The soil profile should be evaluated by a local health department sanitarian or a registered soil scientist to ensure that it is appropriate for wastewater treatment and disposal.

When constructing a septic system it is essential that all components of the soil absorption field be level. If a line lies at too steep a grade or if the distribution system is not level, the wastewater will not be evenly distributed to all portions of the soil absorption field. This may overload one part of the field. The heavy equipment used in home construction can compact the soil. During construction of the house, the area designated for the soil absorption system as well as the required replacement area and the area directly downhill should be fenced off to keep out heavy vehicles. Also, constructing and excavating a system during periods of high soil moisture can result in excessive soil smearing and compaction.

Lack of Maintenance

The septic tank should be pumped about every three years to remove the sludge and scum retained in the tank and prevent clogging of the soil absorption field. More frequent pumping is needed if a garbage disposal is used in the home. Biological and chemical septic tank additives are not necessary and do not eliminate the need for pumping (Mancl, 1998b). A septic tank is equipped with baffles at both the inlet and outlet. The inlet baffle prevents short-circuiting of the sewage and the outlet baffle prevents the floatable scum from moving out into the soil absorption field. In time, these baffles can deteriorate and drop off into the tank. It is a good idea to check the condition of the baffles when the tank is being pumped and replace those in poor condition with sanitary tees.

Corrective Action

Any repair or new installation of a septic system must be approved by the local sanitarian and a permit issued by the local health department.

Water Conservation

This reduces the amount of water the absorption field must accept. It also reduces the flow through the septic tank allowing more time for solids to settle out. Water conservation can prolong the life of any sewage system. Install additional lines to soil absorption field, which increases the capacity of the soil absorption system to accept wastewater.

Install an Alternate Soil Absorption Field

This involves constructing a second soil absorption system and diverting all of the wastewater to it for at least one year to rest the original field. The fields can then be alternated. Alternating soil absorption fields are required for all new installations and are highly recommended as a corrective measure for existing systems.

Repair Physical Damage

Leveling the distribution box or repairing crushed or broken pipe may be necessary to restore the system. Tree roots may be interfering with the operation of the soil absorption field and must be removed.

Improve Surface and Subsurface Drainage

Divert all surface and groundwater away from the soil absorption field. The soil must absorb all the wastewater from the house; surface and groundwater will only add to the load.

Construct a New or Replacement System

In some cases corrective measures are not enough; a new system must be constructed. Do not place more soil over a surfacing soil absorption field; this does not fix the system and it will soon surface again. Do not just pipe the sewage to the road ditch, storm sewer, stream or a farm drain tile; this pollutes the water and creates a health hazard. Do not run the sewage into a sink hole or drainage well; this pollutes the groundwater. Do not wait for the system to fail before pumping the septic tank. Once a system fails it is too late to pump the tank. A properly designed, constructed and maintained septic system can effectively treat wastewater for many years. For more information on septic systems, contact your county Extension office or your local health department.

15.4 System Operation and Maintenance

15.4.1 Observed Conditions

During the course of the sampling on this project the Septic system had no significant maintenance performed on it. On one occasion late in the project an electrician was called out to correct a problem with a ground fault interrupter circuit that provided AC power to the sewage effluent pump. This service call related to sampling equipment and not the system itself.

The residents at this site were observed to be out of town much more frequently than those at the other sites. This is reflected in widely varying water usage rates that were recorded. During the summer months from mid-June through mid-August 1998 the site was inaccessible due to a new gate with an electronic lock that had been installed. During this period combination to the new lock could not be obtained because the homeowners were unavailable.

During sampling of this system it was observed that the wastewater line leading from the house to the septic tank always had water in it when it was opened for sampling. This indicates that the pipe and/or septic tank was not leveled correctly. It is not know whether this was caused by sloppy installation practices or subsequent settling of the soil.

15.5 Reported System Performance

Table 15.3 (Otis, et al., 1980) summarizes septic tank effluent quality. In addition to the tabulated results, bacterial concentrations in the effluent are not significantly changed since septic tanks cannot be relied upon to remove disease-causing microorganisms. Oil and grease removal is typically 70 to 80%, producing an effluent of about 20-25 mg/l. Phosphorus removal is slight, at about 15%, providing an effluent quality of about 20 mg/l total P. Brandes (1978) studied the quality of effluents from septic tanks treating greywater and blackwater. He found that without increasing the volume of the septic tank, the efficiency of the blackwater (toilet wastewater) treatment was improved by discharging the household greywater to a separate treatment disposal system. Factors affecting septic tank performance include: geometry, hydraulic 1oading, inlet and outlet arrangements, number of compartments, temperature, and operation and maintenance practices. If a tank is hydraulically overloaded, detention time may become too short and solids may not settle or float properly.

Table 15.3 Summary of Effluent Data from Various Septic Tank Studies.

Parameter	Ref. (2) 7 sites	Ref. (3) 10 sites	Ref. (4) 19 sites	Ref. (5) 4 sites	Ref. (6) 1 site
BOD_5					
Mean, mg/L	138	138[a]	140	240[b]	120
Range, mg/L	7-480	64-256	----	70-385	30-280
No. Samples	150	44	51	21	50
COD					
Mean, mg/L	327	---	---	---	200
Range, mg/L	25-780	---	---	---	71-360
No. Samples	152	---	---	---	50
TSS					
Mean, mg/L	49	155	101	95	39
Range, mg/L	10-695	43-485	---	48-340	8-270
No. Samples	148	55	51	18	47
Total N					
Mean, mg/L	45	---	36	---	---
Range, mg/L	9-125	---	---	---	---
No. Samples	99	---	51	---	---

[a] Calculated from the average values from 10 tanks, 6 series.

[b] Calculated on the basis of a log-normal distribution.

15.6 Field Trial Results

15.6.1 Flow Characterization

A flow characterization was attempted two times at this site and in both cases we were not successful at obtaining data. Installation of the Campbell Scientific CR500 Data Acquisition System required modifications to the flow event sensing procedure. The electrical wiring of the sewage effluent pump and float switch prevented direct monitoring of pump operation. An attempt was made to substitute AC voltage monitoring with pressure monitoring on the outlet side of the pump. However, this approach was not successful and the data acquisition run on this system produced no usable data.

15.6.2 Hydraulic Analysis

A hydraulic analysis of this system was not performed because no bromide tracer data was available due to the failed flow characterization study.

15.6.3 Water Quality Data Analysis

The septic system was performance tested for 35 weeks from April 29, 1998 through December 22, 1998 with 32 influent and effluent samples collected. There were four sampling days when the site could not be accessed because the residents changed the lock on the road gate. The residents could not be contacted during this four-week period. The average daily flow recorded by the onsite flow meter indicated as shown in Figure 15.3, that flow varied between 3 and 115 gpd over this period with an overall average flow of 52.7 gpd. This is approximately 1/10 of the design flow for the unit. Because of the low average flow, the detention time for the system is approximately 23 days, instead of the typical 2-5 days. This averaged flow over the test period was extremely variable and showed evidence of long periods without occupants. The flow was recorded on the effluent side of the treatment unit.

Temperature data (Figure 15.3) for the site indicated no significant difference between the influent and effluent with mean values of 17.2 and 15.4 °C, respectively. Effluent temperature, reflecting the actual operating temperature of the process, varied from 8 to 21 °C over the study period. These temperature differences can affect the performance of biological treatment process.

The electrical conductivity (EC) reflects the total dissolved solids in a particular water sample. In many cases the change in EC can be an indicator of evaporative processes or the addition of chemicals such as from a water softener, laundry operations, reverse osmosis unit, electroplating, or photo developing processes. Many of these processes may add chemicals that cannot be detected by other measurement techniques or require very specialized and expensive analysis. The EC for influent and effluent samples for the septic tank test site are shown in Figure 15.4 the influent and effluent concentrations were generally very close together, but occasionally would exhibit wide variability. The periodic extreme values produced a 40% variation in the means, with mean values of 1,855.8 and 1,413.3 (m s/cm) respectively. This site has a water softener, and it appears that the brine from the backwash cycle of the softener runs

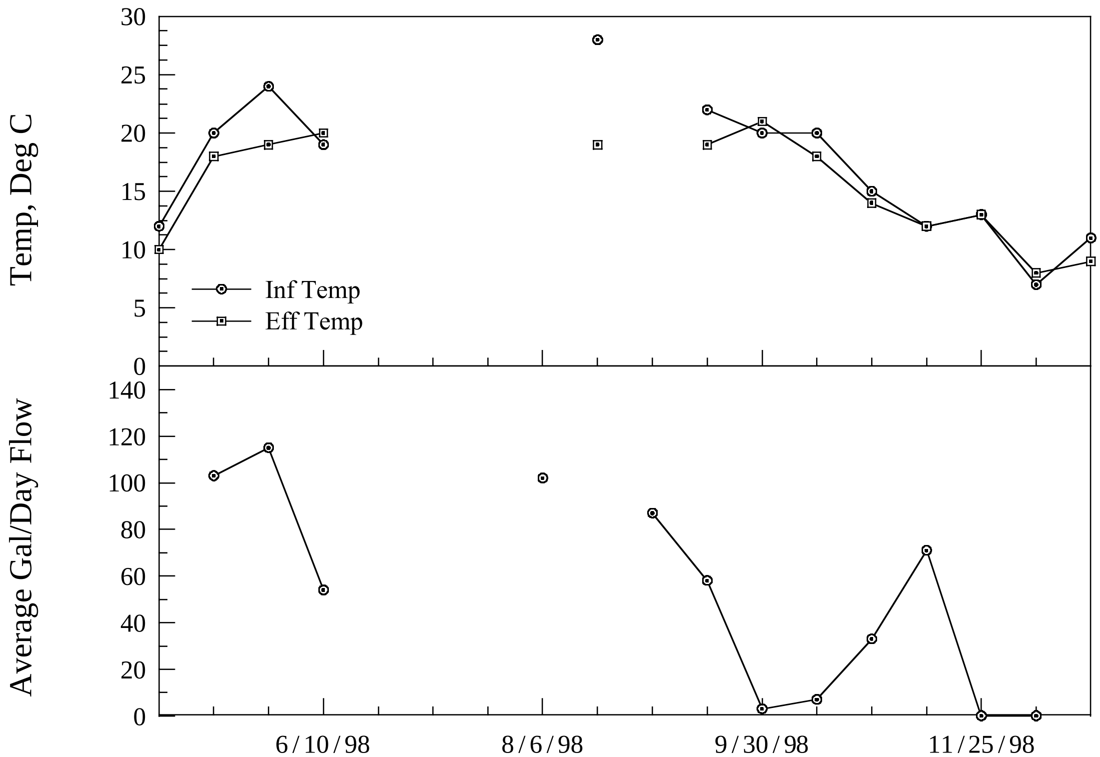

Figure 15.3. Septic System Experimental Data.

through the unit. Typically, water softeners are piped to the waste disposal unit. The plumbing code requires, for all installations, that the regenerate must go to the onsite wastewater treatment unit. This is nonsense for two reasons: the high salt concentration may devastate the microbial populations performing the waste treatment, and the salt in the discharge will not be removed in an onsite wastewater treatment unit. The EC concentrations encountered indicate that the backwash brine was occasionally present in the influent, but due to the detention time in the tank, had not reached the effluent. The influent samples showed greater variability than the effluent with occasional one large spike of 6,700 noted.

Chloride data (Figure 15.4) for the influent and effluent averaged 716.3 and 370.25mg/L respectively. Chloride can be directly contributed by a water softener, with large spikes in concentration, 3,203 mg/L, in the influent observed. A water softener can contribute 20 to 40

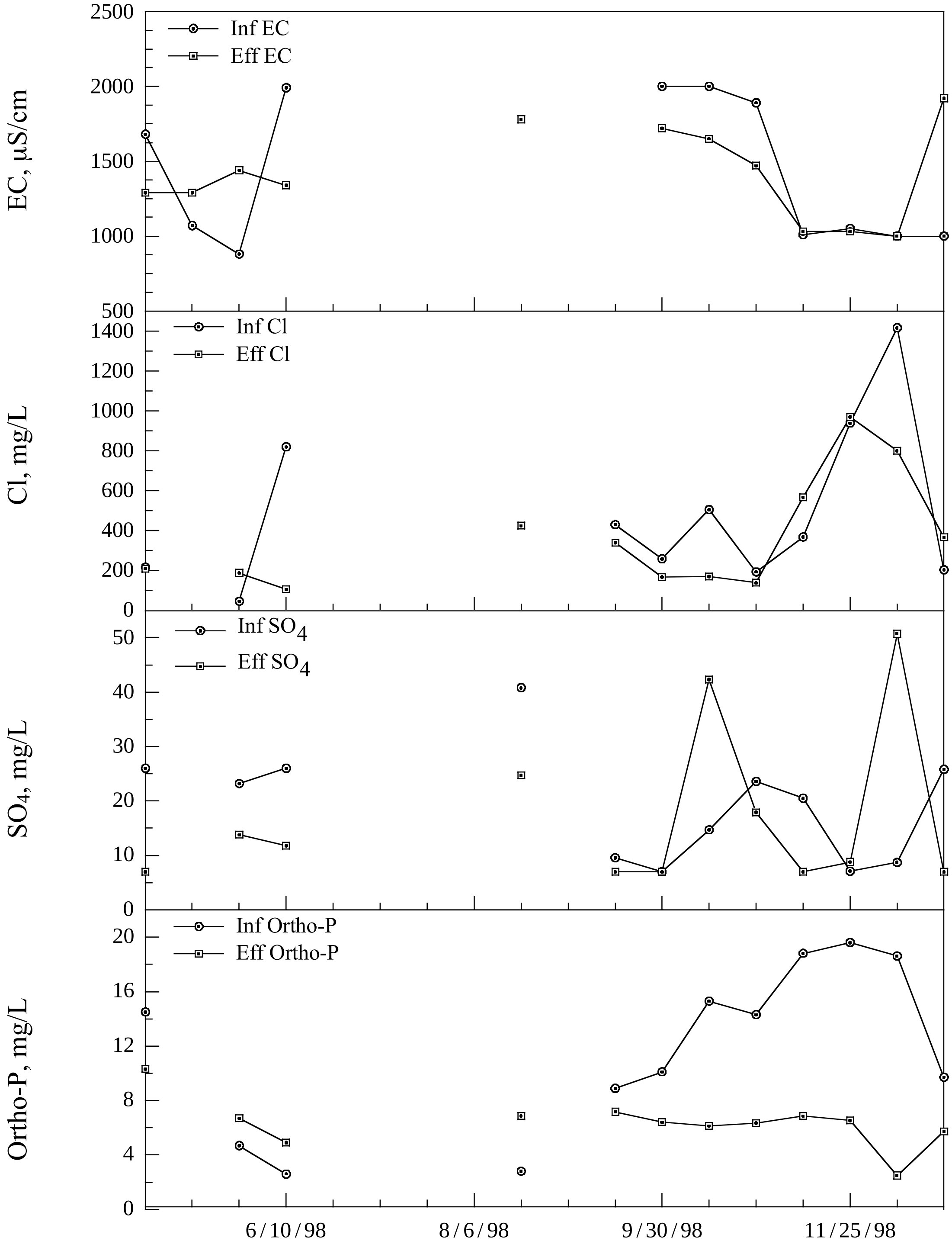

Figure 15.4. Septic System Experimental Data.

gallons in a single backwash cycle with salt concentrations approaching 10,000 to 20,000 mg/L. Potassium salts are also used in home water softener, and would impact the septic system in exactly the same way as sodium salts.

Sulfate data (Figure 15.4), exhibited very little variability in the influent. The influent and effluent sulfate concentrations averaged 19.4 and 17.1 mg/L, respectively and were not significantly different. This indicates that there is probably not enough sulfate in the water to support a microbial population that uses sulfate as its terminal electron acceptor.

The phosphorus concentration in onsite wastewater treatment systems can impact groundwater and contribute to lake and stream eutrophication in many areas of the US. In alkaline arid soils, phosphorus can be readily absorbed within the soil horizon, and is not generally considered a major contaminant. Phosphorus removal in a treatment unit is of interest where phosphorus is a regulated pollutant. The source of phosphorus in household wastewaters can be from human wastes as well as laundry operations. Dissolved or ortho-phosphorus concentrations in the influent and effluent from this test system (Figure 15.4) averaged 11.6 and 6.4 mg/L, respectively. The influent and effluent concentrations were significantly different (p = 0.0088) with a system percent removal of ortho-phosphorus of 44 percent.

The data for pH is shown in Figure 15.5. Maintaining a near neutral pH (6 to 8) is important for the stability of biological processes. Many cleaners and drain openers and other chemicals can drastically raise or lower pH and impact system performance. Influent pH values ranged from 5.9 to 8.36 over the course of the study while effluent values ranged from 6.45 to 7.54. The mean influent and effluent pH values were 7.35 and 7.03, respectively and were not significantly different. While some extreme values were encountered in the influent, the effluent appeared to be much less variable and certainly within range to maintain good biological treatment.

The TSS data values for the system shown in Figure 15.5 indicated some influent events that elevated TSS concentrations over 4,740 mg/L. The average influent concentrations were determined to be 2,232 mg/L with a standard deviation of 1,292. The effluent values averaged 49.2 mg/L with a standard deviation of 20.2. The influent and effluent were significantly different at a p value = 0.0000069. The calculated percent removal TSS was 97.8 percent for this

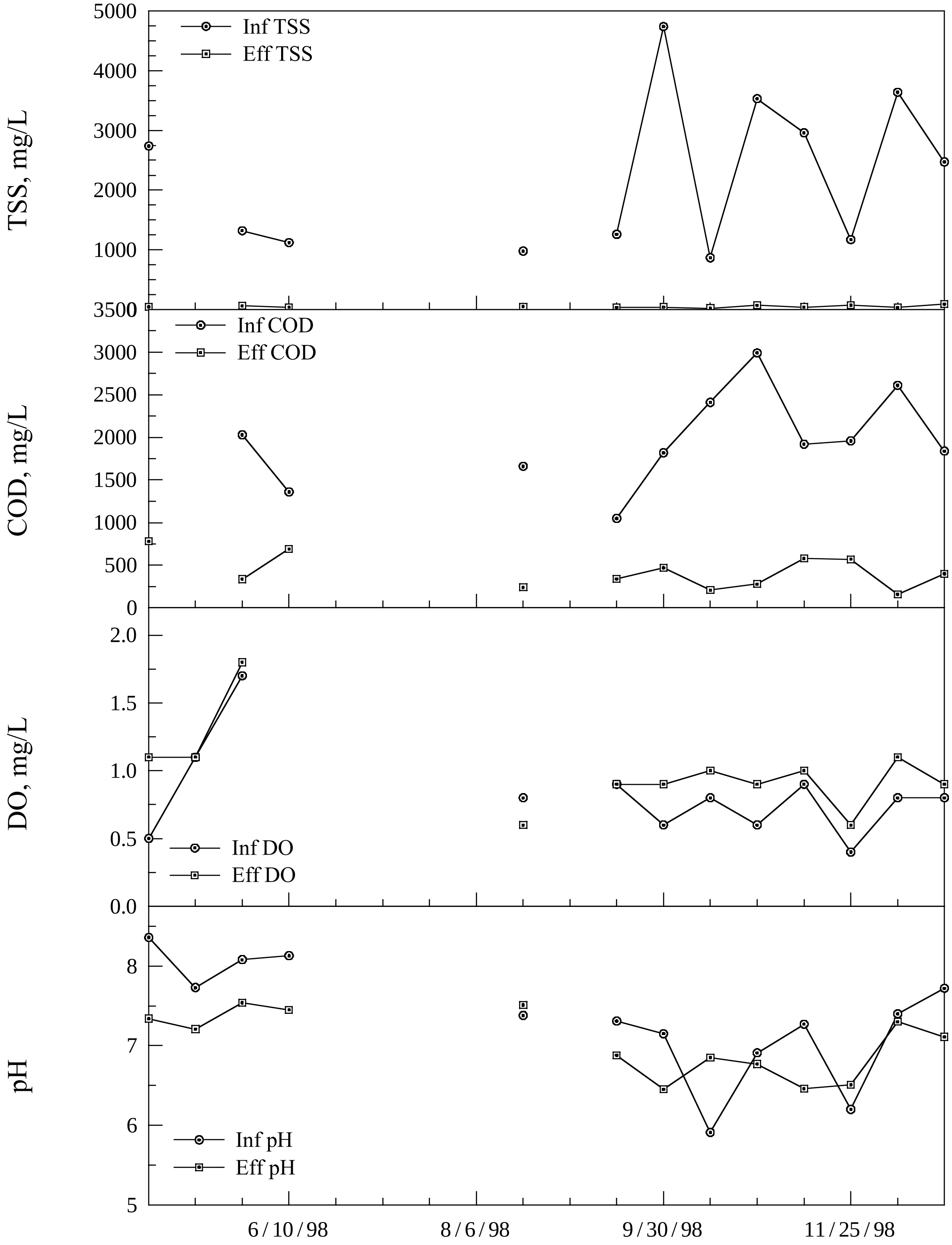

Figure 15.5. Septic System Experimental Data.

system. This removal efficiency is atypically high for a septic tank. Again a reflection of the long detention time in this system. The TSS is consistently below 70 mg/L, but not attaining the 20mg/L standard or less.

The BOD_5 values for this system were not graphed in part because only half the values were provided by the laboratory after samples had been submitted. Many of these were reported as over range or under range and were not usable. The BOD_5 values for the influent had a mean of 1,146.9 mg/L. The BOD_5 values for the effluent had a mean of 150.6 mg/L. The influent and effluent were significantly different at a p value = 0.000194.

The COD data values for the system, shown in Figure 15.5, indicated some influent events that elevated COD concentrations over 7,200 mg/L. These events did not appear to be closely correlated to TSS fluctuations. The average influent concentrations were determined to be 2,404 mg/L with a standard deviation of 1,598 mg/L. The effluent values averaged 421.8 mg/L with a standard deviation of 197.6 mg/L. The influent and effluent were significantly different at a p value = 0.000316. The calculated percent removal of COD was 82 percent for this system.

The DO data values for the system shown in Figure 15.5 indicated some variation in influent DO with concentrations ranging from below 0.4 mg/L to over 1.7 mg/L. The average influent concentrations were determined to be 0.83 mg/L with a standard deviation of 0.34. The effluent values averaged 0.99 mg/L with a standard deviation of 0.31. The influent and effluent were not significantly different at a p value = 0.217. It is the investigators' opinion that the influent and effluent DO levels were probably below 0.5 mg/L. The values measured in the field may have been the result of the sampling protocol. If water has a very low DO, even a brief exposure to air will raise the DO concentration to 1 mg/L. The samples were composited in a manner that may have allowed the necessary exposure.

The ammonia data values for the system, shown in Figure 15.6, indicated a wide variation in influent ammonia with concentrations ranging from 17.9 mg/L to 82.3 mg/L. The average influent concentrations were determined to be 53.47 mg/L with a standard deviation of 22.79. The effluent values averaged 44.42 mg/L with a standard deviation of 14.6. The influent and effluent were not significantly different at a p value = 0.259. The lack of ammonia conversion is expected, since it takes 4.3 mg oxygen to convert each mg of ammonia to nitrate and there is no oxygen available in this system.

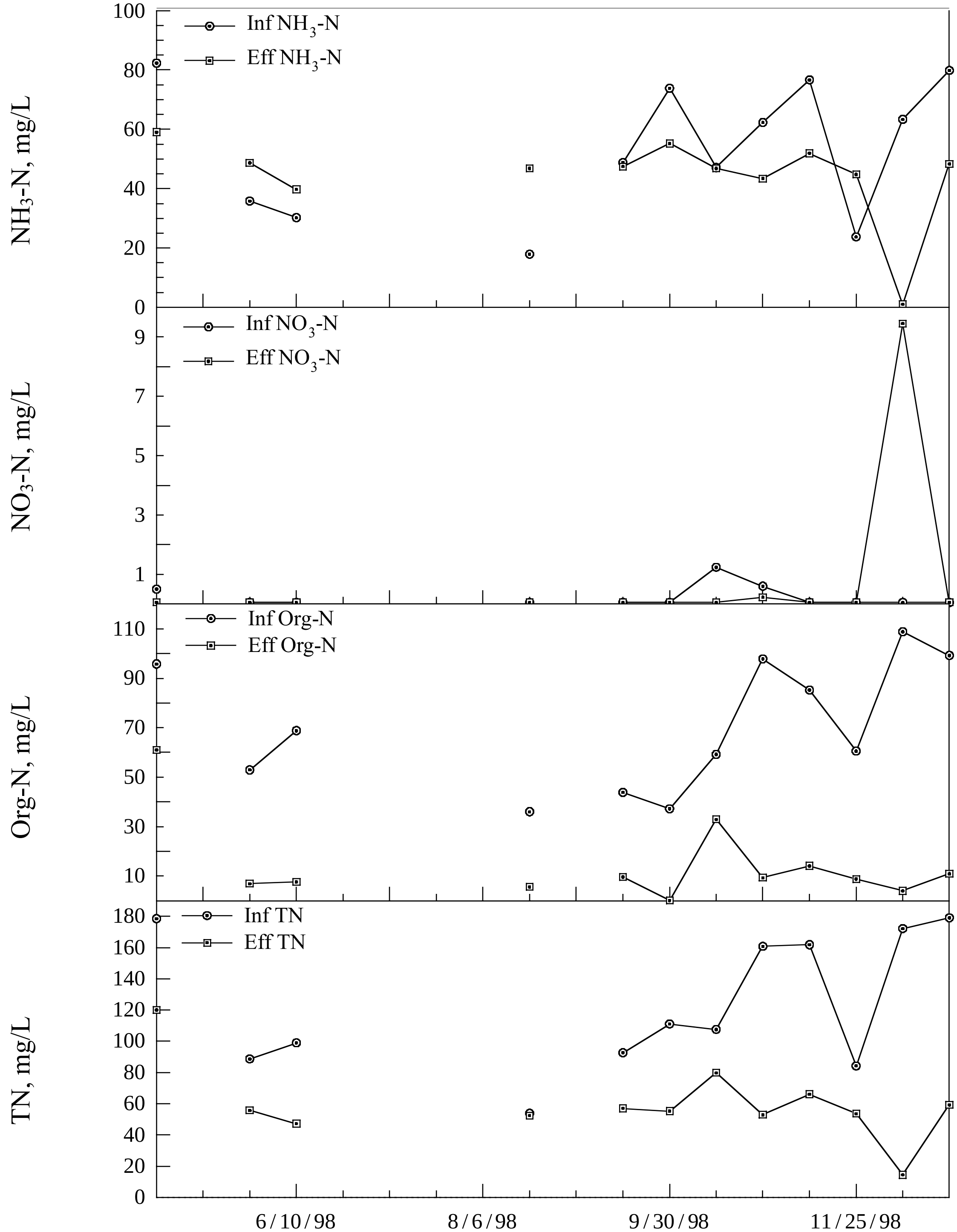

Figure 15.6 Septic System Experimental Data.

The nitrate data values for the system, shown in Figure 15.6, indicated some variation in the influent nitrate, with concentrations ranging from 0.05 mg/L to 1.24 mg/L. The average influent concentrations were determined to be 0.23 mg/L with a standard deviation of 0.37. The effluent values averaged 0.85 mg/L with a standard deviation of 2.71. The influent and effluent were not significantly different at a p value = 0.392. These low nitrate values are to be expected in the septic tank since it is an anoxic system and the nitrate is the electron acceptor of choice for facultative micro-organisms in a low DO environment. It is noted that there appears to be no conversion of ammonia to nitrate, and no net conversion of nitrate to N_2 gas. Traditional nitrification/denitrification is not occurring in this reactor.

The organic-N data values for the system, shown in Figure 15.6, indicated some variation in influent organic-N with concentration ranging from 36 mg/L to over 108.8 mg/L. The average influent concentrations were determined to be 70.43 mg/L with a standard deviation of 26.0. The effluent values averaged 14.23 mg/L with a standard deviation of 16.78. The organic-N may be dissolved, but it will also be associated with biosolids entering the tank. The efficient removal of organic-N is probably related to the high efficiency in TSS removal. Under normal circumstances, we probably cannot expect this level of organic-N removal in a septic tank.

The total nitrogen data values for the system, shown in Figure 15.6, indicated some variation influent total nitrogen with concentration ranging form 53.95 mg/L to 179.05 mg/L. The average influent concentrations were determined to be 124.12 mg/L with a standard deviation of 43.56. The effluent values averaged 59.50 mg/L with a standard deviation of 24.24. Although the effluent TN is fairly high, the septic tank is showing a significant reduction in total nitrogen through the system, a comparison of the influent and effluent mean values yields a p value = 0.000195. Since there is no reduction in nitrate and ammonia, it is speculated that the reduction in total nitrogen is associated with the materials that are settled out and the nitrogen that is converted into cell mass and settled out. Again it is suspected that this level of TN removal is atypical of septic tanks in general.

The fecal coliform (FC) data for the septic system was highly variable with mean influent and effluent values of 4.6×10^5 and 7.3×10^4 cfu/100mL, respectively. If the lowest value is excluded, this resulted in a removal rate of 72.0 percent, which is not even 1 log removal. Influent values ranged from 6.0×10^4 to 1.64×10^6 and effluent ranged from 1 to 3.4×10^5cfu/100mL.

15.7 Conclusions

The septic tank system was performance tested for 35 weeks from April 29, 1998 through December 22, 1998 with 32 influent and effluent samples collected. There were four sampling days when the site could not be accessed because the residents changed the lock on the road gate. The flow characterization and reactor tracer analysis were not performed on the system. The installation, maintenance, and operation of the system were evaluated. The system as tested evaluated performance based on effluent from the house as the influent to the system.

This period with an overall average flow of 52.7 gpd. These flows were significantly less than (approximately 1/10) the design flow (500 gpd) for the unit. Because of the low average flow, the detention time for the system is approximately 23 days, instead of the typical 2-5 days. This averaged flow over the test period was extremely variable and showed evidence of long periods without occupants. The flow was recorded on the effluent side of the treatment unit.

A summary of operating parameters for this system is shown in Table 15.4. This data indicates that the system as tested was operating well during significant portions of the testing.

Table 15.4 Operating and Design Parameters for the Septic Tank System.

Parameter	Loading lbs/day	Effluent mg/L	Percent Removal %
BOD_5	0.50	150.6	86.9
COD	1.0	421.7	82.5
TSS	0.97	49.2	97.8
NH_3-N	0.05	44.4	17
TN	0.02	59.5	52
Ortho-P	0.005	6.6	43.6

The critical regulatory parameters for BOD_5, TSS, TN, and FC are shown in Table 15.5 for the test system. These data indicated that average values generated for the system were well above the recommended performance standards for any of the zones listed. The best data for the field trials are shown in Table 15.5, along with the long term averages and the proposed target levels. The typical septic tank system is the baseline system against which all advanced systems must be compared. If one of the advanced systems cannot achieve a greater BOD, TSS, and TN

Table 15.5 Comparison of Septic Tank System Data and Proposed Performance Standards.

	Field Trial Data		Performance Standards		
Parameter	Overall Mean	Best Results[1]	Zone A	Zone B	Zone C
BOD_5, mg/L	150.6	148.6	30	20	15
TSS, mg/L	49.2	32.6	30	20	15
TN, mg/L	59.5	45	30	20	10
FC, cfu/100mL	73,012	32,820	100	50	1
COD, mg/L	421.7	246	N/A	N/A	N/A
Ortho-P, mg/L	6.6	1.5	N/A	N/A	N/A

[1] Mean of lowest five data points (throwing out the lowest value) with recorded flow event

removal than the septic tank system, it should be considered a failure. Unfortunately, the septic tank system included in this study appears to be atypical. From, BOD, TSS, flow, and nitrogen data collected in this study, it appears that the septic tank system is receiving a very concentrated wastewater stream and has an extremely long residence time. The mean influent BOD and flow measured in this study was 1147 mg/L and 53 gpd respectively. Otis et. al., (1980) report that typical residential averages for BOD influent values in the range of 200 – 290 mg/L and flow is approximately 45 gpcd. Since there are two people at this site, a flow of 90 gpd might be expected. This system had a number of days in which the flows were in the range of 90 gpd, but it also had a number of days when the flow was less than 10 gpd. Clearly, the system is outside of the normal range of values. It appears that there are a significant number of days when the residents are not using water. The low flow has increased the detention time in the reactor from the typical 2-5 days to 23 days. A typical unheated anaerobic reactor with a detention time of three days will reduce the influent BOD by ~50 %. This tank has a 23-day detention time. It is not reasonable to compare the advanced systems to a septic tank system with a 23-day detention time.

Chapter 16 - Surveys

16.1 Survey Design

As in any study of a practical technology, it is necessary to involve those whom the technology will affect. Consumer product manufacturers spend large amounts of their time surveying both current and potential customers to determine their preferences and reactions to what is offered on the market. The present study of onsite wastewater treatment systems is no different, especially when the results may help determine new ordinances. To establish some of the day-to-day conditions on the practical side of onsite treatment systems, it was necessary to survey both the installers and users of these systems.

Two survey forms were developed, one labeled "Installers" and one labeled "Homeowners." (See Appendix for copies of each survey and raw results form). The composition of questions was the product of many meetings between project participants. Draft questions were included, discarded and re-included based on the concerns of the participants that developed over the course of the project. In the design of the survey several factors were taken into account. The first was the need for hard data. For both groups it was desirous to know what kind of systems were being installed in the market place and in what numbers. Maintenance specifics were important numbers for both groups. Information on household plumbing specifics and water use habits was important data to get from the homeowners while information dealing with onsite system failures were details needed from the installers.

Also important were the views and opinions of the two groups to the various issues presently being discussed about onsite treatment. For the Installers some of the areas of interest included: training on alternate systems by the manufacturers, quality of available septic tanks, recommendations on pumping, perceptions on leachfield failures, and opinions on the City/County organization. The Homeowner survey contained questions dealing with training by installer, opinions on the performance of the onsite systems in protecting the groundwater, thoughts on maintenance, and like the Installer's survey, perceptions of the City and County.

It was the opinion of the authors that a critical aspect of any survey is its length. If it is too long then the respondent tends to lose interest and fairly quickly becomes annoyed. The two surveys used were designed to be administered in about ten minutes. The Installer's survey originally contained 25 questions, though two questions were not posed to the respondents in a majority of the cases because it was found that the questions were either not easily understood or too difficult to answer. The Homeowner survey contains 22 questions.

The next step was deciding who and how many to call. For the Installers, the "who" was decided by taking a look at the Septic Tank and Septic Maintenance listings provided by the Bernalillo County Department of Environmental Health (BCDEH), the Albuquerque and Surrounding Communities Yellow Pages, and the listings for Septic Services provided on the Internet. For the Homeowners, a list of those who made applications for onsite treatment permits in the last three years was provided by BCDEH. It was determined, in a large percentage of the cases, that phone numbers and owners of the systems had changed from the time of the permit application to the present. In order to call the houses that corresponded to the location of the onsite system, it was necessary to access a database that allowed phone number searches based on addresses. Standard services available on the Internet were not up to the task so a special database was purchased. Powerfinder by PhoneDisk, a software on CD, had the necessary search protocols. Notwithstanding, there were a large number of addresses that had no corresponding phone numbers. This greatly reduced the available pool of possible survey participants.

For the Installers survey, a sample of 50 businesses were chosen from the available lists. Effort was put into finding companies that installed or repaired onsite systems as a main part of their business. This versus predominately plumbing companies that would only work with onsite on occasion. These surveys were conducted by phone during regular working hours over a 10-day period from the 18 through the 27 of January, 1999.

With the Homeowners survey the number of people queried was controlled more by the availability of phone numbers than any other factor. After many hours spent with the Powerfinder software and a highlighting marker, a list of 100 phone numbers were selected that most likely corresponded to onsite system owners. The surveys were conducted by phone during the evening hours from 5 p.m. to 8 p.m. over a 5-day period from the 15th through the 19th of February, 1999.

16.2 Results

For the Installers survey there were 20 respondents out of a total of 50 clients contacted on a response of 40% of the sample population. The main reasons noted by the survey takers for those not responding were that the business had either closed, the respondent stated they were too busy to respond, or calls were not returned. Assuming a normal statistical distribution, a sample size of 20 implies a plus or minus 10% error on the results with a 90% confidence interval.

In the case of the Homeowners survey, 48 out of the 100 or 48 percent of system owners called responded to the survey. The majority of the remaining phone numbers called were either disconnects or those answering were not willing to take the survey. A sample size of 48 implies a plus or minus 7% error on the results with a 90% confidence interval.

Survey responses were compiled using the EPI-INFO software, available over the internet from the Centers for Disease Control (CDC) Atlanta, GA. The results of both surveys are tabulated on the following pages. Except where indicated the tabulated results are in percentages of respondents answering each of the questions. For Question #2 and Question #7 of the Installers survey, the results are totals of all those responding. As noted in the design section, two questions were removed from the Installers survey, #19 and #20, because they proved impractical to ask. Unless indicated next to the tabulated results, it is implied that all respondents answered the stated question (see Question #9, for example, on the Installers survey).

16.3 Discussion of Results

As borne out by the results, the search for businesses that install or repair large numbers of onsite systems was successful. Those companies who responded to the survey had installed over 1,000 new systems and had maintained (usually pump-outs) nearly nine thousand systems in 1998 alone. The vast majority of the new systems were traditional septic tanks (98%). There were, however, a relatively large number of alternative leachfield systems installed. Alternative systems accounted for only 2% of the systems installed in the past five years.

Installers indicated that the most common reason for an installation was new construction (75%), system failure (19%) and remodeling (6%). Leachfield failures (94%) were by far the most common reason for repairs. Septic tank failure was cited by only 6% of the respondents. Seventy one percent of the businesses contacted provided maintenance services for onsite systems.

Of those respondents who installed alternative treatment systems 75% stated they received adequate technical support for installation and maintenance, but the respondents were split (50% - 50%) as to whether the company provided training in the form of course work, videos, or seminars. Seventy five percent of the respondents felt that onsite systems did not receive enough maintenance, and 94% felt that systems were not pumped out enough or only when there was a problem.

Eighty one percent of the installers surveyed, thought the quality of septic tanks they used were good. Nineteen percent thought septic tanks were of average or poor quality. Installers were split about if anything could be done to improve septic tank quality, but a majority agreed (75%) that a septic tank certification program would improve septic tank quality.

Surprisingly, 63% of the installers felt the existing ordinances provided sufficient guidelines for design and installation of leachfields, eighty one percent of the installers surveyed felt that conventional leachfields were adequate to protect the environment and public health. Significantly 44% of the installers felt the reason for leachfield failure was bad design and 13% attributed failure to improper installation. Twenty five percent thought inadequate pumping caused leachfield failures. But only 41% of the installers supported a mandatory pumping schedule for onsite systems.

Finally, to the credit of the City and County regulators, 76% of the installers were aware of the efforts to develop a new ordinance for onsite wastewater treatment. Another interesting item from the point of view of the survey takers was the overwhelming support for the County. Sixty nine percent of those interviewed thought that the existing ordinances were well-designed and well-implemented. Interestingly enough, most were also aware that new ordinances were being developed, but thought that an ordinance requiring septic tank pump-out would not be effective.

The responses from the Homeowners survey closely reflected those from the Installers survey. Eighty eight percent of those who responded either had conventional septic tanks or did not know what they had. The majority (68%) thought that maintenance was important, but the responses as to how often a septic tank should be pumped out were varied. While 60% thought that the standard time periods of one or two years were best, nearly 40% had very different ideas: from seven years to never. The importance of clean water was clearly demonstrated as 92% responded saying that it was very important to them. Seventy four percent also thought that the City/County were doing a good job in terms of protecting drinking water and the environment. Many respondents were quick to point out that the technician from the East Mountain Field Office was "one of the most capable employees that they ever had the pleasure know."

Eighty nine percent of the homeowners did not know the manufacturer of the system, but 8% of the systems were owner installed. Seventy three percent of the installed systems were estimated to cost less than $5,000, but 11 % were estimated to cost more than $5,000. About 68% of the homeowners stated they performed regular maintenance on their systems. Fifty eight percent described maintenance as pumping, followed by yeast addition (17%), bacteria (9%), and enzymes (15%). Maintenance was performed yearly by 48% of the respondents, monthly (22%), and surprisingly (9%) performed weekly maintenance.

Only 5% of the homeowners had lived in their house for less than six months and 59% lived in their house for 4 years or more and 36% had an operating water softener. Of these with a water softener, 86% stated that it treated the entire household water supply and 85% of these systems discharged to the septic tank.

An overwhelming 83% of the homeowners did not have a wastewater system maintenance manual and 84% did not receive any training regarding operation of the system.

Eight five percent of the homeowners had never had any trouble with their system. Problems encountered with the system included; odors (25%), standing water (25%), and clogging (25%). These problems appeared not to occur at any regular time of the day, month, or year. Fifty three percent of the homeowners stated they had their septic system pumped every 6-12 months, but 39% had never had their tank pumped. Average cost for maintenance was $115/year.

Finally, 82% of the homeowners thought these systems adequately protected their drinking water supply, 92% believed that having clean water to drink or cook with was important, and 74% believe the County and City were doing a good job of protecting drinking water supplies and the environment.

Chapter 17 - Summary and Recommendations

17.1 Summary

The overall conclusions of the report are summarized in this section. These conclusions are combined with sections on specific guidelines for various aspects of onsite systems.

The Whitewater system, a sedimentation unit and a suspended-growth, aerobic biological process, was tested for 65 weeks. This was an “off the shelf” unit without a pretreatment trash tank. The system was hydraulically tested, exhibited no short-circuiting and had near complete-mix flow, flow pattern. It was difficult to determine from unit operations perspective, how this system achieved denitrification. The system influent and effluent data is shown in Tables 17.1 and 17.2 for concentrations of BOD_5, COD, TSS, FC, ammonia, and TN, respectively. This system achieved an average percent removal of 71.6, 59, 98, 91, 49.4, and 68.3 % for BOD_5, COD, TSS, FC, ammonia, and TN, respectively. The best effluent values for this system were 36.3 mg/L, 181.9 mg/L, 8.4 mg/L, 2.7×10^4 cfu/100mL, 1.5 mg/L, and 14.2 mg/L for BOD_5, COD, TSS, FC, ortho-P, and TN, respectively. The unit did appear to nitrify/denitrify the wastewater. Operationally the system had problems with the supply of air to the process and the aeration fittings appeared to be a weakness in the system.

The submerged surface flow constructed wetlands system, a fixed-film, anoxic/aerobic/anaerobic with emergent plants biological process, was tested for 65 weeks. The SSF cell of the system was hydraulically tested and exhibited no short circuiting and a dispersed plug-flow, flow pattern. The system influent and effluent data is shown in Tables 17.1 and 17.2 for concentrations of BOD_5, COD, TSS, FC, ammonia, and TN, respectively. This system achieved an average percent removal of 53.7, 42, 45, 95.6, 43.5, and 43.8 % for BOD_5, COD, TSS, FC, ammonia, and TN, respectively. The best effluent values for this system were 23.6 mg/L, 97.8 mg/L, 2 mg/L, 1.31×10^3 cfu/100mL, 1.5 mg/L, and 14.2 mg/L for BOD_5, COD, TSS, FC, ortho-P, and TN, respectively. Operationally the system worked very well and for much of the summer months the wetland cell produced no discharge. The overall system with a septic tank, two-stage wetlands (SSF and SF cells) operated at a zero discharge mode throughout the study period.

Table 17.1 Summary of Mean Influent Data for the Five Test Systems.

Parameter/System	Whitewater	Clearstream[1]	Septic Tank	FAST	Wetlands
BOD_5, mg/L	330	N/A	1,147	523	208
COD, mg/L	914	N/A	2,404	1,842	544
TSS, mg/L	1,296	N/A	2,232	1,356	41
NH_3-N, mg/L	28	N/A	54	49	70
TN, mg/L	62	N/A	124	69	77
FC, cfu/100mL	6.02×10^5	N/A	4.6×10^5	6.02×10^5	3.43×10^5
Ortho-P, mg/L	4.9	N/A	6.4	10.1	6.7

[1]Very few data values, see section on Clearstream for summary.

Table 17.2 Summary of Mean Effluent Data for the Five Test Systems.

Parameter/System	Whitewater	Clearstream[1]	Septic Tank	FAST	Wetlands
BOD_5 , mg/L	93.5	N/A	150.6	76.0	96.0
COD , mg/L	374.5	N/A	421.8	882	310.0
TSS , mg/L	26.2	N/A	49.2	28.3	16.7
NH_3-N , mg/L	13.9	N/A	44.4	16.0	39.6
TN, mg/L	19.7	N/A	59.5	29.0	43.2
FC, cfu/100mL	5.25×10^4	N/A	7.3×10^4	2.04×10^4	1.48×10^4
Ortho-P, mg/L	3.0	N/A	6.4	3.3	4.8

[1]Very few data values, see section on Clearstream for summary.

The Clearstream system, a suspended-growth, aerobic biological process with recirculation to an anoxic suspended growth chamber, was operated only intermittently as the homeowner moved out in the early part of the study. This unit was a custom designed system. The system was not hydraulically tested and insufficient data was collected to evaluate the system.

The Co-FAST or FAST system, a suspended-growth combined with fixed film, aerobic/anoxic, biological process, was operated for 55 weeks. This was an off the shelf unit without a pretreatment trash tank. The system was hydraulically tested and exhibited no short circuiting and had a near complete-mix flow, flow pattern. The system influent and effluent data is shown in Tables 17.1 and 17.2 for concentrations of BOD_5, COD, TSS, FC, ammonia, and TN, respectively. This system achieved an average percent removal of 85.5, 52.1, 97.9, 99, 67.5, and 75.5 % for BOD_5, COD, TSS, FC, ammonia, and TN, respectively. The best effluent values for this system were 60 mg/L, 593 mg/L, 8.8 mg/L, 2.0×10^2 cfu/100mL, 1.6 mg/L, and 13.3 mg/L for BOD_5, COD, TSS, FC, ortho-P, and TN, respectively. Operationally the system had problems with the supply of air to the process, the PVC pipe supplying air was not properly glued and leaked decreasing the air applied during a portion of the study.

The septic system, a sedimentation unit and anaerobic biological process, was operated for 35 weeks over the study period. This was an off the shelf unit without a pretreatment trash tank. Two attempts were made to hydraulically test the unit, but failed due to insufficient flow. The occupant appeared to be out of town frequently or worked away from the home for extended periods. The system influent and effluent data is shown in Tables 17.1 and 17.2 for concentrations of BOD_5, COD, TSS, FC, ammonia, and TN, respectively. This system achieved an average percent removal of 86.9, 82, 97.8, 72, 17, and 52 % for BOD_5, COD, TSS, FC, ammonia, and TN, respectively. The best effluent values for this system were 148.6 mg/L, 246 mg/L, 32.6 mg/L, 3.28×10^4 cfu/100mL, 1.5 mg/L, and 45 mg/L for BOD_5, COD, TSS, FC, ortho-P, and TN, respectively. Operationally the system was typical for septic tanks systems, but exhibited a very low overall flow.

A summary of the operational parameters and loadings for all the systems is shown in Table 17.3. This data indicates that all systems tested were hydraulically underloaded with respect to design assumptions. Note that for all processes the actual HRT was much longer than the design values. These longer values can have both a negative and positive impact the performance of all systems tested. While loadings for BOD, TSS, and TN vary, they are similar despite the wide range of households tested under this study.

The systems tested in this study did not appear to be capable of meeting a 10 mg/L TN standard for effluent discharges to the subsurface. This does not mean there are no systems that can do this, but under the conditions used in this study none of the test sites were able to meet this standard. In order to meet the standard for TN, systems should be optimized for N removal and carbon removal rather than just carbon removal as is the case for most systems currently used. The business of onsite wastewater treatment system is very dynamic and several states are

Table 17.3 Summary of Operational Data for the Five Test Systems.

Parameter/System	Whitewater	Clearstream	Septic Tank	FAST	Wetlands[1]
Design Flow, gpd	500	N/A	450	450	450
Design HRT, days	1.81	N/A	2.7	1.8	2.7
Actual Flow, gpd	161.8	N/A	52.7	119.6	201.1
Actual HRT, days	5.6	N/A	23.0	6.8	8.0
BOD Loading, lbs/day	0.52	N/A	0.50	0.52	0.34
TSS Loading lbs/day	1.74	N/A	0.98	1.35	0.05
TN Loading lbs/day	0.08	N/A	0.05	0.12	0.13

[1] Loadings do not include raw waste but effluent from a pretreatment septic tank.

adapting nitrogen limits in their regulations and ordinances. Given time to develop, new, lower-cost, technologies will be commercially available that will meet more stringent standards. Aeration systems (blowers or compressor, airlines, and diffusers) associated with the mechanical aerobic units were a noticeable weakness and the partial or full failure of this part of the system resulted in very poor system performance. Rigorous inspections must be performed and air supply units pressure-tested to assure their operability. Solids wasting in aerobic systems (biosystems) needs to be performed on a regular basis, as well as regular maintenance (every 6 months pump 200 gallons and check system components);

Systems must be evaluated based on both the theory of operation and manufacturer supplied test data. In many cases the theory of operation is largely ignored and the claims published by the manufacturer are the only substantial information available. Test data must be collected in a systematic fashion to allow for the analysis of the data. With nitrogen, the complete species must be given not just nitrate or ammonia or TKN. The conditions (composite grab or otherwise) under which the data was taken and the flow must be specified. Without information of this kind, rational judgement cannot be made about the ability of a system to meet the required standards.

Surveys conducted on sample populations of installers and homeowners in the Bernalillo County area provided interesting results on a number of topics. Significant results from the installers survey indicated that Septic tanks accounted for nearly 98% of all installations and alternative systems accounted for only 2% of the installations. Installers indicated the most common reason for an installation was new construction (75%), followed by system failure (19%), and remodeling (6%). Leachfield failures were the most common reason for repairs and poor installation or drainfield undersizing cited as the most common reason for failure. Eighty nine percent of the installers thought the quality of septic tanks they used was good, but 75% thought a septic tank certification program would benefit their business. Seventy six percent of the installers were aware of the efforts by the city and county to improve the onsite wastewater ordinances. Homeowners surveyed indicated that 68% of the respondents had maintenance performed on their systems and 53% had their septic tanks pumped every 6 to 12 months. But 39% had never had their systems serviced or pumped. Finally, 74% percent of those homeowners surveyed thought the county was doing a good job at protecting drinking water supplies and the environment.

Finally, based on this study and the data gathered from other states, various literature sources, and previous phases of work, guidelines covering the use of alternative systems were developed. These guidelines provided specific recommendations pertaining to lot sizes, site evaluations, system performance standards (technology based standards and best management practices (BMP)), system maintenance guidelines, system installation, and inspection of new and existing systems. These are given in detail in the following section.

17.2 Recommendations

Groundwater is used by the majority of the people residing in the Bernalillo County/Albuquerque area for water supply. The overall protection of this resource is the basic goal of the guidelines provided. Onsite wastewater treatment provides many residences with the only form of protection from disease and impacts of poor water quality for their water supplies. All onsite treatment systems must be properly designed, manufactured, installed, and maintained. Onsite treatment systems consist of a treatment unit and disposal system. Treatment units provide for the removal of waste components from wastewater discharges from residential dwellings. The disposal system provides for the ultimate disposal of treated wastewater through discharge to the ground or use by plants, or total reuse for water supply.

17.2.1 Lot Size Determinations

The recommendations for lot sizing in Bernalillo County, New Mexico should focus on site specific criteria, such as the risk of aquifer contamination with bacteria and pathogens from septic tank effluent, the soil characteristics of the area, and the prescribed setbacks from wells, water lines, etc. However, if the risk of aquifer contamination is determined to be low, the soil characteristics are satisfactory and all the prescribed setbacks are met, the minimum lot size recommended is one acre. Many studies have shown that lot sizes less than one acre are inadequate for soil treatment of septic tank effluent. In addition, the recommended distance of 200-foot minimum from the nearest septic tank effluent source to a well, in most cases can only be accommodated in a one acre size lot (Michael J. Bitner & Timothy Graves, and others, 1992).

17.2.2 Site Evaluation

The following is the recommended procedure to be used for determining whether a site of a specified size will adequately handle the effluent from a conventional septic tank installation: Note: It is recommended that the site evaluation, be conducted by a person certified as an official "site or soil evaluator". An individual must take and pass (75% score) a site or soil evaluator exam in order to be issued a certificate. Persons meeting the following criteria will qualify to take the exam: New Mexico Registered Sanitarians, New Mexico Registered Professional Engineers, Engineers in Training, etc. The exam can be modeled after existing exams in other states like Massachusetts, Texas, and Arizona.

- Determine if site is located in a high septic tank density area (more than 320 systems per square mile, (Bitner, Graves, et al, 1992);

- Assess ability of soil to treat septic tank effluent based on: a) log of soil formations determined by a minimum of two test holes dug in close proximity to the proposed drain field location, b) percolation test results, and c) seasonal high ground water level, also determined by test hole analysis, soil color using the Munsell system, and the use of USGS wells, etc.;

- Identify and evaluate the topography of the proposed disposal area based on the following criteria: a) presence of bedrock outcrops or areas with many stones and/or boulders, b) steep slopes (greater than 3:1, horizontal to vertical), c) flat low-lying areas adjacent to surface water bodies, etc.;

- Determine if the area is in a flood zone or its proximity to one;

- Determine the hydrogeologic properties of the proposed disposal area with respect to the following: a) estimated direction of water flow, b) estimated depth to bedrock, c) drainage classification of dominant soil type, d) location of every water supply, public and private, e) lateral distance to surface water, etc.;

- Determine if the proposed disposal area is in a pollutant (nitrate) sensitive area based on the following: a) ambient aquifer water quality (nitrates) based on historical data or current water quality data from a sampling well if available. If the applicant decides not to submit aquifer water quality data or there are no monitoring wells in the vicinity, and there is no agricultural activity in the area, a disposal limit of 800 gpd/acre is recommended. If there is agricultural activity in the area, a disposal limit of 400 gpd/acre is recommended. The previously recommended disposal limits only apply if soil, topographic and hydrogeologic conditions are met.

If one or more of the above site evaluation criteria are not met, the applicant should not be allowed to install a conventional septic tank system with a typical soil absorption leach field as specified in the current regulations. However, the applicant may propose to use an alternative onsite wastewater disposal system, which removes nitrate. If the system proposed has been proven effective by some predetermined criteria (e.g. past performance in similar situations, pilot testing data, etc.), the applicant may receive an individual permit to install an alternative onsite wastewater disposal system. It is recommended that a database be compiled of alternative systems that have proven effective in total nitrogen (TN) removal in other states in similar applications. It is implied that if a system is capable of reducing the concentration of TN in the effluent, the BOD5, TSS and other pollutants will also be considerably reduced.

Once the database has been compiled, alternative systems should be classified as approved for general use, remedial use (upgrade or replacement of failing systems), and provisional or piloting use (more information needed to assess system performance). However, prior to system installation the applicant (owner) must agree to the following:

- Provide monthly influent and effluent analysis results from a specified laboratory for the first year of system operation and quarterly influent and effluent analysis (BOD, TSS, and TN) results for the second year;

- Maintain the system as specified by the system manufacturer; a manufacturer's representative must be located within the state of New Mexico for systems classified for general use,

- Allow inspectors to inspect system on a periodic basis.

17.3 Performance Standards

Performance standards will be established by two methods: best management practices (BMP) and treatment standards. Best management practices are a series of guidelines for technical and management options. Treatment standards of system performance will be technology-based standards (TBS). Technologies for use in onsite systems can consist of many different types of processes including physical/chemical and biological approaches. Each technology will be considered as a unit operation and will be evaluated on a mass flow basis. The burden of proof of a performance claim is placed upon the manufacturer to comply with and supply the necessary data.

17.3.1 Technology Based Standards

Regions of sensitivity should be established within the county by using DRASTIC or similar type risked-based assessment modeling. DRASTIC takes into account parameters such as ground slopes, depth to groundwater, depth to bedrock, permeability and other factors to develop areas of vulnerability to surface inputs of contamination. This modeling effort will establish clearly delineated regions. Regional maps should be resolved to a scale necessary to resolve boundaries between each region. These regions will be used to determine the degree of treatment performance needed to assure protection of groundwater. These regions provide general guidelines only, we feel that further site delineation is still required to assure protection. In addition, as site information is gathered the database on conditions within the county expands.

Based on the DRASTIC modeling, a minimum of four zones of risk associated with treatment performance will be defined as outlined in Table 17.4. Treatment units must be certified for each zone by providing performance data. This data must be developed using an acceptable test method and acceptable methods of analysis for water quality parameters. The data must be developed in accordance with procedures outlined by the National Sanitation Foundation (NSF) for onsite systems. NSF outlines procedures for flow, PH, BOD, and TSS testing that include shock loading and shut down conditions. In addition, testing for total nitrogen (organic nitrogen, nitrate nitrogen, and ammonia nitrogen) must be completed for influent and effluent to meet the conditions of this guideline. All tests must be conducted during the same time period and on the same system. All other conditions specified by NSF must be met.

Table 17.4 Proposed Performance Categories for Onsite Treatment Systems.

Parameter/Category	Zone A	Zone B	Zone C	Zone D
Septic Tank	Yes	n/a	n/a	n/a
TSS, mg/L	60	30	10	5
BOD, mg/L	60	30	10	5
TN, mg/L	60	50	25	10
MPN, cfu/100mL	1,000	1,000	100	<1

All water quality analysis must be conducted using the most recent edition of the methods outlined in Standard Methods for Examination of Water and Wastewater published by the American Water Works Association. An alternative to this test method is the procedure outlined as follows:

- One or more systems of the same make and model can be tested on a site anywhere. The manufacturer must provide site and process drawings of the installed systems. And include information about the original design of the system including sizing information for flow and waste strength.

- The system must be equipped in a way that will allow for taking grab samples of influent and effluent samples. Drawings of these sample ports must be provided with the information from the previous section.

- After the system has been established for period of no less than six months of continuous operation sampling can be begin.

- Sampling consists of taking influent and effluent grab samples from the sample ports These samples must be taken in clean, plastic or glass bottles, preserved using methods outlined in Standard Methods and transported to an accredited laboratory for analysis. Samples must submitted for analysis in an acceptable time period after samples were collected (24 hrs max with preservation). The effluent sample port must be equipped with a pump and flow measurement device (Small water meter) to measure accumulated instantaneous flow.

- Field measurements should consist of flow data (averaged for 1 week period) and water quality data (pH, temperature, ORP, and DO).

- Collected samples will be analyzed for BOD, TSS, ammonia nitrogen, nitrate nitrogen and organic nitrogen using method outlined in Standard Methods.

This sample procedure will be repeated for four weeks with sampling at one week intervals, thus providing four replicate data sets. All data will be submitted on forms provided by the county and must include all information requested in these procedures. In addition, several other conditions must be met in order for a particular technology to be accepted. The technology must:

- have state-based authorized factory representatives,
- provide installers with documented training materials and workshops,
- provide performance data for specified equipment and process units,
- provide maintenance materials and training for installers,
- provide service contract options for installers for equipment,
- provide ongoing support for all aspects mentioned above, and
- support the local onsite wastewater association.

Once the technology has established this information with the County by formal application, the technology is approved for installation under the categories designated in Table 17.1. This approval is interim pending the completion of the second step in the process. This step involves the installation and testing of five units of the technology. These units must be the same as the technology in the application process and any modifications provided must be approved by a review by the county. Interim status allows the authorized representative to install five units. These units must then pass a field performance test verifying the stated system performance. The authorized representative must be willing to participate in this final step or further system installation will be denied. This step of the testing will begin three months after installation is complete and will be continue for six months to allow for several sets of samples to be taken.

The field performance testing consists of the installation of temporary sample sumps in the unit flow. These sumps must be located in a way that allows sampling of the influent and effluent from the system. In addition, a flow measurement device such as a pump coupled to a standard water meter must be included. Data from the flow meters must be collected once every two weeks. Water quality samples must be taken once every two months for six months from the sample sumps. These samples at the expense of the technology representative will be analyzed for pH, BOD, TSS, nitrate nitrogen, ammonia nitrogen, and organic nitrogen. All testing will be conducted in accordance with the most recent editions of Standard Methods.

The sample data and flow information for all test systems must be compiled and presented as a part of the application process. This data will be compared to the existing performance data. The information will become part of the public record for that system. Based on this information the County will approve or deny the system for final certification.

In addition, to the information provided from manufacturers, the county should also use the following procedures to assure that the units under review can meet performance indicated by test data.

- For biological systems the units must provide adequate initial solids removal similar to a primary clarifier in large scale systems. This unit must reduce the organic loading to the main treatment unit of the overall process. The detention time of the unit should be adequate to account for the flow and storage separated solids. In addition the unit should also separate oil and grease. The plumbing of this unit should be similar to a conventional septic tank. For many processes this unit is called a trash tank and may be fabricated by the manufacturer or may be a small septic tank specified to be a part of the process. A minimum of a four hour detention time is recommended and storage of solids sufficient for three years of storage.

- Aerobic units should convert BOD to CO_2 and water and Ammonia to nitrate. There has to be oxygen available to do this and sufficient detention time to retain the solids (biomass). Typically these units must operate like an extended aeration reactor thus detention time should be long (> 8 hrs). BOD reduction can accomplished in less time, but nitrification usually requires a much longer time period. The air supplied must not only provide the oxygen for the biological processes but must provide mixing. Assuming BOD and ammonia nitrogen numbers similar to the EPA onsite guide oxygen requirements can be calculated, the aerator should supply an adequate amount of air to meet the requirements for treatment and mixing. Typically BOD and ammonia demand are nearly equal thus oxygen for ammonia conversion cannot be ignored.

- A final clarifier must be included in the unit. This clarifier retains solids produced by the bacteria in the aerobic unit. This operation also protects the drainfield from clogging.

- The only method for ultimate removal of nitrogen from these systems is by denitrification. This process requires that anoxic conditions be developed and the effluent be exposed to this environment long enough to allow nitrate to convert to nitrogen gas. This process does not take as long as the conversion of ammonia to nitrate. Usually some sort of upflow unit with media to support a bacterial group is provided. This last step must be present in the flow scheme in order for final nitrogen removal to take place.

17.3.2 Best Management Practices

The BMPs consist of a set of guidelines that will enhance or preserve system performance. These guidelines do not specify effluent concentrations of various contaminants, but suggests methods to reduce contaminants discharge into onsite systems. Onsite system are vulnerable to shock loadings from salts or toxic compounds and there is an operator to change or alter the operation to adjust to the influent loading. Eliminating factors and minimizing discharges are the most practical approaches to insuring minimal impact from these sources.

The following is a proposed list of BMPs for the Bernalillo county area.

- A system requiring that septage pumping be performed at set intervals should be established to insure that septic tanks are pumped. Table 17.5 shows one method of estimating frequency of pumping. While this may be a good guide for information we feel that setting a time of three to five years would be better to insure groundwater protection. The pumping frequency should be linked to the categories determined via DRASTIC modeling. Maintenance can be required through contracts, operating permits, and local ordinances/utility management. Local governments can issue renewable operating permits that require users either to have a contract with an authorized inspection/maintenance professional or to demonstrate that inspection and maintenance procedures have been performed on a periodic basis.

Table 17.5 Estimated Septic Tank Pumping Frequencies in Years for Residences (from Ohio extension material)

Tank Size (gal.) — **Household Size (Number of People)**

	1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**
500	5.8	2.6	1.5	1.0	0.7	0.4	0.3	0.2	0.1	-
750	9.1	4.2	2.6	1.8	1.3	1.0	0.7	0.6	0.4	0.3
1000	12.4	5.9	3.7	2.6	2.0	1.5	1.2	1.0	0.8	0.7
1250	15.6	7.5	4.8	3.4	2.6	2.0	1.7	1.4	1.2	1.0
1500	18.9	9.1	5.9	4.2	3.3	2.6	2.1	1.8	1.5	1.3
1750	22.1	10.7	6.9	5.0	3.9	3.1	2.6	2.2	1.9	1.6
2000	25.4	12.4	8.0	5.9	4.5	3.7	3.1	2.6	2.2	2.0
2250	28.6	14.0	9.1	6.7	5.2	4.2	3.5	3.0	2.6	2.3
2500	31.9	15.6	10.2	7.5	5.9	4.8	4.0	4.0	3.0	2.6

- Ongoing efforts for educating homeowners via mailings or other avenues should be conducted. This information should address the importance of maintenance, and other aspects. Material should be provided that is part of the seller package of real-estate information that addresses how to care and maintain a septic tank. A form should be developed that is signed during closing of all real estate transactions that involves informing the new owner that they now have and must maintain an onsite system. Many of the problems associated with improper use of septic systems may be attributed to lack of user knowledge on operation and maintenance. Educational materials for homeowners and training courses for installers and inspectors can reduce the incidence of pollution from these widespread and commonly used pollution control devices.

- Garbage Grinders: Eliminating the use of garbage disposals can significantly reduce the loading of suspended solids, nutrients, and BOD to septic systems, as well as decreasing the buildup of solids in septic tanks, thus reducing pumping frequency. Garbage grinders should be eliminated from all households that use onsite treatment systems.

- Organic Solvents: No solvent or organic compound such as paints, cleaning compounds, etc. listed by USEPA as hazardous substance should be discharged into a septic tank system.

- Water softeners should be plumbed to treat only water necessary to minimize blowdown discharge and to minimize impacts of increased sodium on the drainfield. Blowndown discharges should be directed to a French drain outside the dwelling. The blowdown should not be allowed to discharge to the onsite treatment system. Equipment suppliers should be contacted to discuss plumbing options.

- Cottage industries or hobbies that perform activities such as electroplating, dark room developing, jewelry making and others should not discharge wastes from these operations to the onsite treatment system. The county should establish a list of activities that must take precautions. This list should be distributed with all permits, reapplication, or real estate transactions. Materials from these activities should be captured in separate containers and taken to County sites for appropriate disposal. Education of photo suppliers and hobby sales outlets as to proper disposal is suggested (this has worked well in the area of mercury batteries).

- Excessive quantities of bleach or high phosphate detergent should not be used in septic tanks. These materials upset treatment operations and can clog drainfields.

- Additives sold as enhancers for improving septic tank performance or reducing odors should only be allowed if they are on the a list of approved substance maintained by the county. Massachusetts onsite regulations further provide that the following is a list of septic system additives that have been allowed for use, with certain conditions, as it has been determined that the product will not harm the septic system components, or adversely affect system function or the environment when used on a schedule recommended by the manufacturer.

It is important to stress that the Department's determination to allow the use of an individual constituent is not an endorsement or approval with respect to the benefit, effectiveness, or performance of the system additive. As additional additives are evaluated and allowed usage is granted by the Department, this list will be updated.

They then provide a list of which only a portion is given as follows:

- Bio Rem St (septic system additive) Caldwell Environmental; Contact person - Robert Caldwell, 978/266-1221 or 1-800-370-0077

- Bio Rem Gt (soil absorption system conditioner/restorative) Contact person - Robert Caldwell, 978/266-1221 or 1-800-370-0077.

- Septic Zest (septic system additive) Analab Inc., 59 Davis Ave Norwood, MA. Contact person - Mr. Keith Marshall.

- Trap Zap Plus (septic system additive, soil absorption system conditioner/ restorative) Trap Zap Environmental Inc., P.O. Box 8619, 59 Lee Ave, Haledon, N.J. 07538-8619. Contact person - E. Charles Hunt, President.

- LS-1472 (septic system additive) AquaTerra Biochemical Corporation of America, 1917 Lancaster Hutchins Road, P.O. Box 496, Lancaster, Texas 75146. Contact person - Carolyn Seroka, Regulatory Specialist, 214/438-0857.

- Advanced Formula Rid-X (septic system additive) Reckitt & Colman, Inc. 225 Summit Ave, Montvale, N.J. 07645-1575.

- Ultra Rid-X (septic system additive) Reckitt & Colman, Inc. 225 Summit Ave, Montvale, N.J. 07645-1575.

- Aid Ox (septic system additive) Cloroben Corporation, 1035 Belleville Tpk, Kearny, N.J. 00732. Contact person - John Wrobleski.

- BIO-REM E-D (septic system additive) Cape Cod Biochemical Co., P.O.Box 990, Pocasset, MA 02559. Contact person - Rick Howe.

- CCLS (septic system additive) Cape Cod Biochemical Co., P.O.Box 990, Pocasset, MA 02559. Contact person - Rick Howe.

- Septic Helper 2000 (septic system additive) Miller Plante, Inc., P.O. Box 2117, Cliffside Park, N.J. 07010. Contact person - Herb Miller, President.

- Microbe/Lift (septic system additive) Ecological Laboratories, Inc., 70 N. Main Street, Freeport, N.Y. 11520. Contact person - Barry Richter.

17.4 Suggested Maintenance

Trash Tanks

Trash tanks are a pre-treatment unit for the advanced treatment systems. The purpose of the trash tank is similar to the primary clarifier in a conventional municipal treatment plant. This unit is to remove the large settleable material, which will reduce the loading on the aerobic unit by as much as 40%. This unit should be checked periodically for sludge, oil, & grease accumulation. In the event that there are excessive amounts of either material accumulated in the tank, it should be pumped.

Main Unit

Each of the proprietary units will have a unit specific maintenance schedule that will be provided by the manufacturer. The following are general guidelines suggested for all units:

- Periodically check the soil around the units for leakage. Symptoms of leakage will include surfacing and excessive growth.
- Check aerator nozzles for plugging.
- Moving parts need to be checked for wear and lubricant applied as needed.
- Check for excessive bio-solids buildup.
- Check for erosion adjacent to the treatment unit.
- Be aware of odor development and if odor develops check the blower and aerator.

In addition to the above, any media in the tank has to be checked for deterioration, and replaced as needed.

Furthermore, the following maintenance and operation guidelines should be followed by septic system owners to avoid trouble, (these guidelines should be distributed to homeowners receiving a permit to construct a septic system):

- DO have your tank pumped out and system inspected every 3 to 5 years by a licensed septic contractor.
- DO keep a record of pumping, inspections, and other maintenance.

- DO practice water conservation. Repair dripping faucets and leaking toilets, run washing machines and dishwashers only when full, avoid long showers, and use water-saving features in faucets, shower heads and toilets.

- DO learn the location of your septic system and drainfield. Keep a sketch of it handy for service visits. If your system has a flow diversion valve, learn its location, and turn it once a year. Flow diverters can add many years to the life of your system.

- DO divert roof drains and surface water from driveways and hillsides away from the septic system. Keep sump pumps and house footing drains away from the septic system as well.

- DO take leftover hazardous household chemicals to your approved hazardous waste collection center for disposal. Use bleach, disinfectants, and drain and toilet bowl cleaners sparingly and in accordance with product labels.

- DO NOT allow anyone to drive or park over any part of the system.

- DO NOT make or allow repairs to your septic system without obtaining the proper permits from the department.

- DO NOT use commercial septic tank additives. These products usually do not help and some may hurt your system.

- DO NOT use your toilet as a trash can by dumping non-degradables down your toilet drains. Also, do not poison your septic system and the groundwater by pouring harmful chemicals down the drain. They can kill the beneficial bacteria that treat the wastewater.

Alarms

All electrical devices associated with the system should have appropriate failure alarms. The minimum units requiring alarms are the pumps and blowers. The pumps must have at a minimum high water and failure alarms. These must be both audible and visual. The alarms need to be checked periodically to ensure that they continue to be functional.

Electrical

The electrical circuit associated with the treatment unit should be a ground-fault interrupt system. This system should be tested to insure that the interrupt function is operational.

17.5 Guidelines for the Installation of Systems

17.5.1 Installers

All installers in the county should be required to participate in a county installer certification program. Only licensed installers will be allowed to perform professional system installations. In addition, an installer must certify that they have been factory trained and certified to install the alternative unit being installed. In the event that they are not factory trained and certified, they must have a trained responsible party onsite supervising the installation. The installer is the ultimate responsible party and is responsible for the successful completion of all system testing during installation. It is the installer's responsibility to provide training for the homeowner. The actual training may be provided by the manufacturer's representative, but the installer is the responsible party. The training must be intense enough so the homeowner can manage the system without relying on the installer. It is also the responsibility of the installer to provide a detailed written O&M manual to the homeowner. The manual must be specific to the system installed at the site.

17.5.2 System

Prior to county approval of installation for a factory system, there will be a designated state or county representative for the manufacturer. It is unacceptable to have a disinterested party installing treatment hardware. Providing a designated state or county representative is not sufficient to claim the privilege to install. Any alternative system must have county approval prior to installation. A minimum criteria for approval is NSF certification. Any tanks installed must meet appropriate ASTM standards, either for septic tanks or fiberglass tanks. The system must be installed and inspected. Inspection must occur prior to backfilling. The tanks must be installed in an acceptable level manner. All piping must be continuous and water tight. All tanks must undergo hydraulic testing. All systems shall provide for the separation of water softener regenerate. This is in apparent contradiction to the UPC. However, removal of this high-strength saltwater stream from the system is critical to the success of the system. The regenerate water will inhibit the biological activity in the tank.

Septic Tanks and other units including trash tanks, pump sumps, aerobic treatment tanks, and drain fields will be leveled using acceptable surveying equipment. This equipment will include surveyor's transits or portable lazer type levelers. Rule of thumb, line site estimation and leveling with water tubing is not acceptable. All alternative systems must provide a suitable hose bib adjacent to the treatment system.

Electrical

All electrical work must be either performed by a licensed electrician, inspected by a licensed electrician, or inspected by an electrical inspector. If a contractor performs the work an electrical inspector must review the finished site. A ground-fault interrupt 110 volt duplex circuit will be provided adjacent to the reactor in all alternative systems. This is provided for maintenance and cleanup convenience.

Drainfield

Drainfields both standard and alternative should be designed based on three percolation tests and soil test pits. The percolation tests and the pits can be coincident. The design should be based on minimizing risk. Use of NRC's soil maps is unacceptable. If existing topsoil, subsoil, peat or other unsuitable or impervious soil layer above the requisite four feet of naturally occurring material is found to be ineffective for drainfield use, an adequate fill material may be utilized. However, all soil absorption systems constructed in fill shall be sized using the soil type of the underlying naturally pervious material.

17.5.3 Inspection Guidelines

Inspection guidelines should focus on inspection of new systems while they are being installed, as well as the inspection of existing systems to assess system performance and to prevent system failure. Inspectors should be certified or licensed by the State of New Mexico. The existing regulations for the state of New Mexico governing the installation of new onsite wastewater treatment systems are adequate for inspection of new systems, however, they do not provide for the inspection of existing systems. Therefore, the following guidelines will concentrate on the inspection of existing systems to insure compliance with the state's environmental regulations.

17.5.4 New System Inspections

New system inspections should focus on making sure that all the system components, including the soil absorption system (drainfield), are in place, are sized correctly and that the actual system location meets all the setback requirements. The system evaluation (testing), on the other hand, after installation should include water-tightness tests on tanks and pipes, making sure all electrical components have power and are working properly, etc.

17.5.5 Inspection of Existing Systems

Inspection of existing onsite wastewater treatment systems, whether conventional or alternative, is not a simple task. In fact some states (Minnesota, Massachusetts, etc.) have developed extensive inspection protocols to facilitate this task. Although difficult, the task of inspecting existing systems has become necessary, since a large number of these systems are believed to be failing in their function of providing an effluent free of pollutants and pathogens. Due to the exorbitant costs associated with the implementation of a statewide program to inspect every individual onsite wastewater treatment system, it is recommended that existing onsite systems be inspected when the property of its location is sold and also when a permit to add dwellings or modify an existing dwelling on the property is submitted. An inspection should consist of the collection and recording of the following information:

- A general description of the system components and layout,
- Quantification of the source/type of septage; this should include the design flow and whether the facility was occupied at the time of inspection,
- Water use records for the previous 2 years if available,
- A description of the septic tank, including age, size, condition, design, thickness of grease/scum layer, depth of the sludge layer and distance of sludge to outlet tee, evidence of leakage into or out of the tank, etc.

- Characterization of the distribution box, and of dosing tanks with pumps, if any including: 1) evidence of solids carryover, 2) leakage into or out of box, 3) is flow equally divided, and 4) any evidence of backup,

- A description of the condition of the soil absorption system including the following: 1) any signs of hydraulic failure, 2) condition of surface vegetation, 3) level of ponding within disposal area, 4) encroachments into disposal area, and 5) other sources of hydraulic loading.

Criteria Exhibited by Potentially Failing Systems

- There is backup of sewage into the facility served by the system or any component of the system as a result of an overloaded and/or clogged soil absorption system;

- There is discharge of effluent directly or indirectly to the surface of the ground through ponding, surface breakout or damp soils above the disposal area;

- The static liquid level in the distribution box is above the level of the outlet invert;

- The septic tank requires pumping more than four times a year;

- The septic tank is made of metal, unless the owner can provide proof that the tank was installed within the twenty year period prior to the date of inspection.

Chapter 18 - References

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Chapter 19 - Appendices

APPENDICES A - G

FINAL REPORT VOL. II
ALTERNATIVE ONSITE WASTEWATER TECHNOLOGIES

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